Project Report

Putah Creek Bridge Rehabilitation at Stevenson Bridge Road
Federal Project No. BRLS 5923(059)
Bridge No. 23C0092

Prepared For:

Solano County Resource Management
Public Works Engineering

Prepared By:

QUINCY ENGINEERING

January 2018

REGISTERED PROFESSIONAL ENGINEER
Greg Young
No. 67707
Exp. 6/30/19
CIVIL
STATE OF CALIFORNIA

Prepared by:

Greg Young

Greg Young, Project Engineer

Reviewed by

Greg Young	1-18-2018	(916) 368-9181
Greg Young, Project Manager Quincy Engineering, Inc.	Date	Telephone

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APPENDIX

EXECUTIVE SUMMARY

This report summarizes the seismic analysis and recommended rehabilitation strategy of the existing Putah Creek Bridge at Stevenson Bridge Road (Stevenson Bridge). The existing bridge is a four-span concrete bridge with the second and third spans consisting of a reinforced concrete tied arch span (each being 108 feet long). The approach spans (first and fourth spans), consist of reinforced concrete T-beam girders 40 feet long. The bridge was built in 1923 and is approximately 296 feet long by 24 feet wide. Foundations consist of spread footings at the abutments and concrete columns with curtain walls supported by pile footings and the piers.

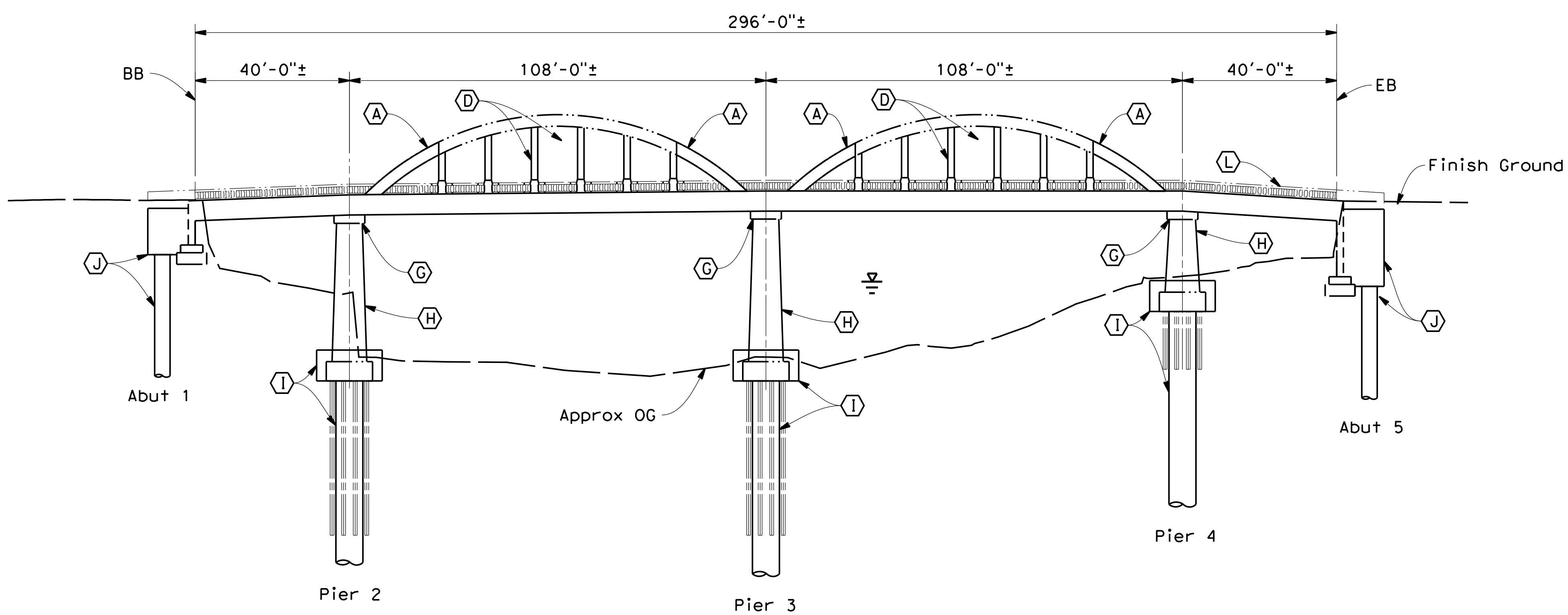
Quincy Engineering, Inc. (Quincy) has been commissioned to evaluate the seismic and scour vulnerability of the existing bridge as well as improve the approach roadway alignment. Quincy has evaluated multiple bridge rehabilitation alternatives, and will develop the Plans, Specifications, and Estimate for the construction of the preferred rehabilitation alternative. This report is also supplemented by previous evaluations of the structure conducted by TRC Ibsen in 2006 and 2007. The PS&E will be further supported by a hydraulic analysis (by WRECO), geotechnical recommendations based on new test borings (by Cal Engineering & Geology), and a detailed site assessment consisting of visual inspections, borescope observations, ground penetrating radar scanning and concrete core testing (by Alta Vista). All supplemental reports and studies (with the exception of the TRC Ibsen reports) are located in the Appendix of this report.

Results from the seismic assessment indicate that most of the arch span members are severely deficient and are incapable of resisting forces from the design earthquake. To meet current design standards, the bridge must be able to remain standing after the design earthquake, defined as a 975-year return period earthquake. In other words, the bridge should be able to withstand an earthquake that has 5 percent probability of occurrence in a 50-year period. For the Stevenson Bridge site, the nearby Great Valley faults could produce an Earthquake up to a 6.7 Maximum Magnitude. Due to its deficiencies, the Stevenson Bridge may collapse under a much lower earthquake that could occur on a much more frequent basis.

The bridge has also been classified by Caltrans as scour critical, meaning it is vulnerable to collapse during extreme flows in the creek. This finding has been supported by the independent scour analysis conducted by WRECO. Consequently, significant retrofitting of the existing structure is required to make the bridge resilient to both seismic and high flow scour events in the creek. The County has previously evaluated and rejected both the bridge replacement and the do nothing alternatives. Therefore, the bridge will be rehabilitated and strengthened.

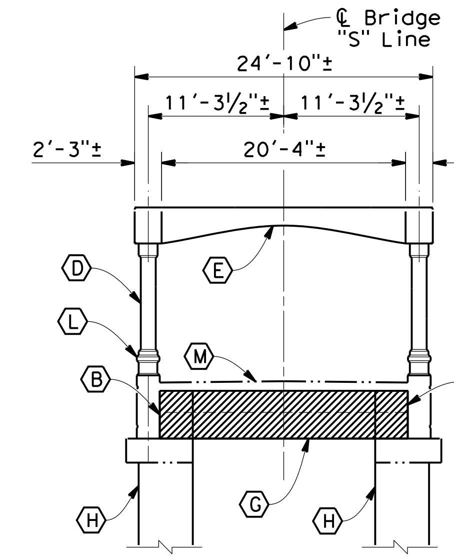
In addition to seismic and scour deficiencies, many repairs are required just to restore the bridge back to the As-built condition. Alta Vista performed in depth mapping and testing in order to determine where repairs are required. The majority of repairs involve removing unsound concrete, cleaning and painting exposed reinforcing steel to prevent further deterioration, and patching with new concrete.

Given the seismic and hydraulic vulnerabilities, multiple rehabilitation alternatives were identified and evaluated. The proposed project will consist of installing new CIDH piles at each support, fiber wrapping of Arch, Portal, Vertical, and Pier members, reinforced concrete Tie Girder bolsters, removal and patch of unsound concrete, epoxy injection of larger cracks, deck methcrylate to seal minor cracks, repair/replacement of the existing concrete railing, installation of Rock Slope Protection and roadway approach improvements. The proposed retrofit is shown graphically in the attached planning study. The estimated project construction cost including roadway improvements are $10,213,000.



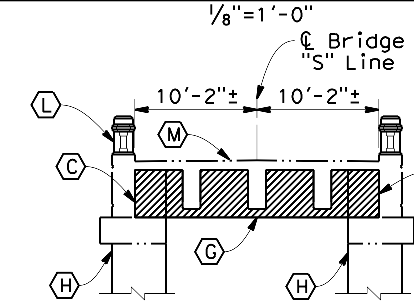
ELEVATION (K)

1"=20'



ARCH SPAN TYPICAL SECTION (K)

⅛"=1'-0"



APPROACH SPAN TYPICAL SECTION (K)

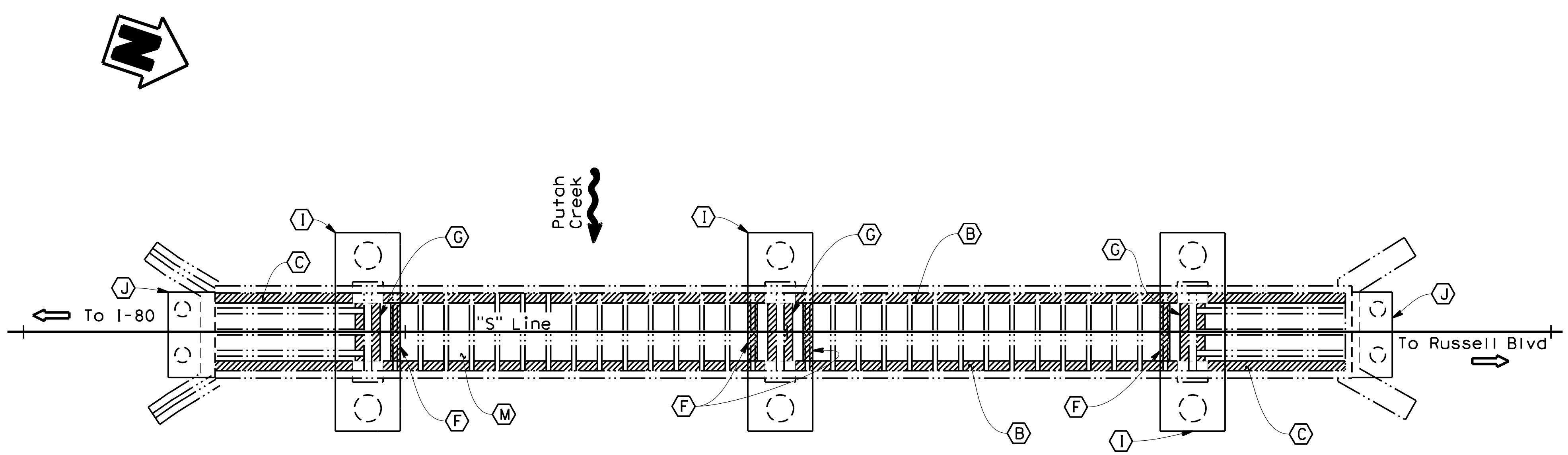
⅛"=1'-0"

RETROFIT LEGEND

- (A) Arch Rib - Fiber Wrap (Spring line to first vertical hanger)
- (B) Tie Girder - Bolster
- (C) Approach Span Exterior Girder - Bolster
- (D) Vertical Hanger - Fiber-Warp or Replace
- (E) Portal Bracing - Fiber Wrap
- (F) Floor Beam - Bolster
- (G) Pier Cap - Bolster
- (H) Pier Column - Remove Internal Pier Wall and Fiber Wrap Columns
- (I) 7' Diameter CIDH Pile and Pier Footing Buildout
- (J) 4' Diameter CIDH Pile and Abutment Diaphragm Bolster behind Existing Abutment
- (K) Remove Unsound Concrete and Patch Spalls
- (L) Concrete Railing - Remove and Reconstruct
- (M) Deck - Epoxy Inject Cracks and Methacrylate

LEGEND:

- Indicates Bolster
- Indicates New Construction
- Indicates Existing structure
- Indicates Highwater elevation
- Indicates Direction of Flow
- Indicates Direction of Traffic



PLAN (K)

1"=20'

REVISIONS				FIELD BOOK NO.	SCALE			SOLANO COUNTY	APPROVED BY:	PUTAH CREEK BRIDGE REHABILITATION	DATE
NO.	DESCRIPTION	APPROVED BY	DATE		HORIZONTAL: AS NOTED	DRAWN BY:	CHECKED BY:	TRANSPORTATION DEPARTMENT		ON STEVENSON BRIDGE ROAD	1-4-2018
					VERTICAL: AS NOTED	SUBMITTED	R.C.E. No.	333 SUNSET AVE. SUITE 230 SUISUN CITY CA 94585 TEL: (707) 421-6069 FAX: (707) 429-2894	DATE:	GENERAL PLAN	SHEET 1 OF 1
											DWG

1. INTRODUCTION

Project Description

The County of Solano (County), in conjunction with the County of Yolo, the California Department of Transportation (Caltrans), and the Federal Highway Administration (FHWA), are proposing to rehabilitate the Putah Creek Bridge (23C0092) at Stevenson Bridge Road. The County also desires to improve the southern approach roadway geometry. The bridge spans the County line between Solano and Yolo Counties, on Stevenson Bridge Road approximately a half mile north of Putah Creek Road.

The Highway Bridge Program (HBP) will provide 88.53% of the funding for this project; however, the County will be responsible for 11.47% local matching funds.

Quincy Engineering, Inc. (Quincy) will complete the PS&E for the roadway and bridge rehabilitation. Professional services will also include structure assessment, preliminary engineering, hydraulic studies, and geotechnical studies.

The purpose of this project is to improve public safety by rehabilitating the seismically vulnerable and scour critical structure. Additional safety features include improving the roadway alignment, and repair of the existing concrete railing.

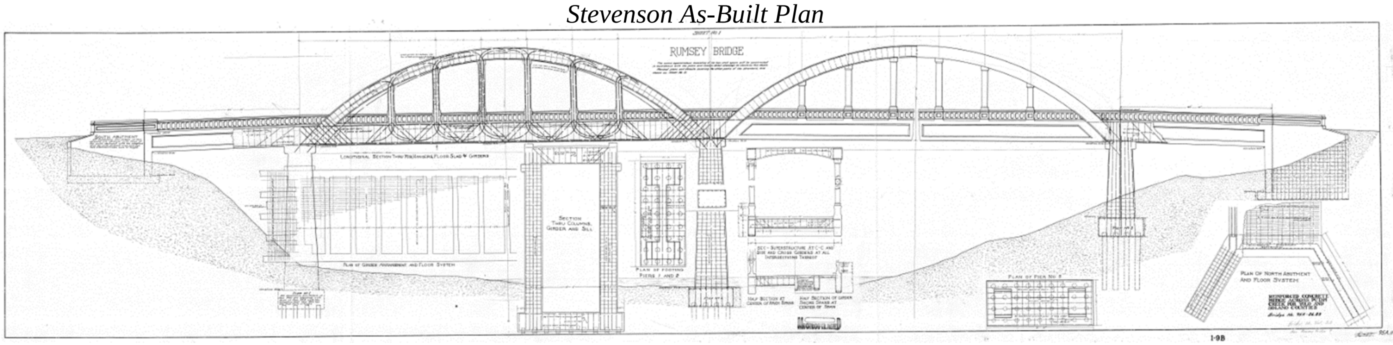
Depending on environmental and permit constraints, construction of this project is anticipated to be completed within one or two construction seasons.

This Project Report summarizes preliminary engineering completed to date, and includes site-specific data such as topographic surveys, geology, hydraulic, and environmental information. This Project Report will also define the bridge design criteria and preferred alternative to be used in the final design PS&E phase.

Existing Structure

Putah Creek Bridge (23C0092) at Stevenson Bridge Road was constructed in 1923. The existing roadway is functionally classified as a major collector (based on Caltrans CRS maps), which provides access for approximately 789 vehicles per day (2008 ADT from 2015 BIRIS report) between Solano and Yolo Counties. The structure is comprised of reinforced concrete T-beam approach spans and concrete tied arch main spans. The bridge structure is approximately 296 feet long and 24 feet wide with two 40-foot long approach spans and two 108-foot long tied arch main spans. The substructure is supported on reinforced concrete piers with curtain walls, founded on timber and concrete pile foundations. The abutments are founded on spread footings.

Stevenson As-Built Plan

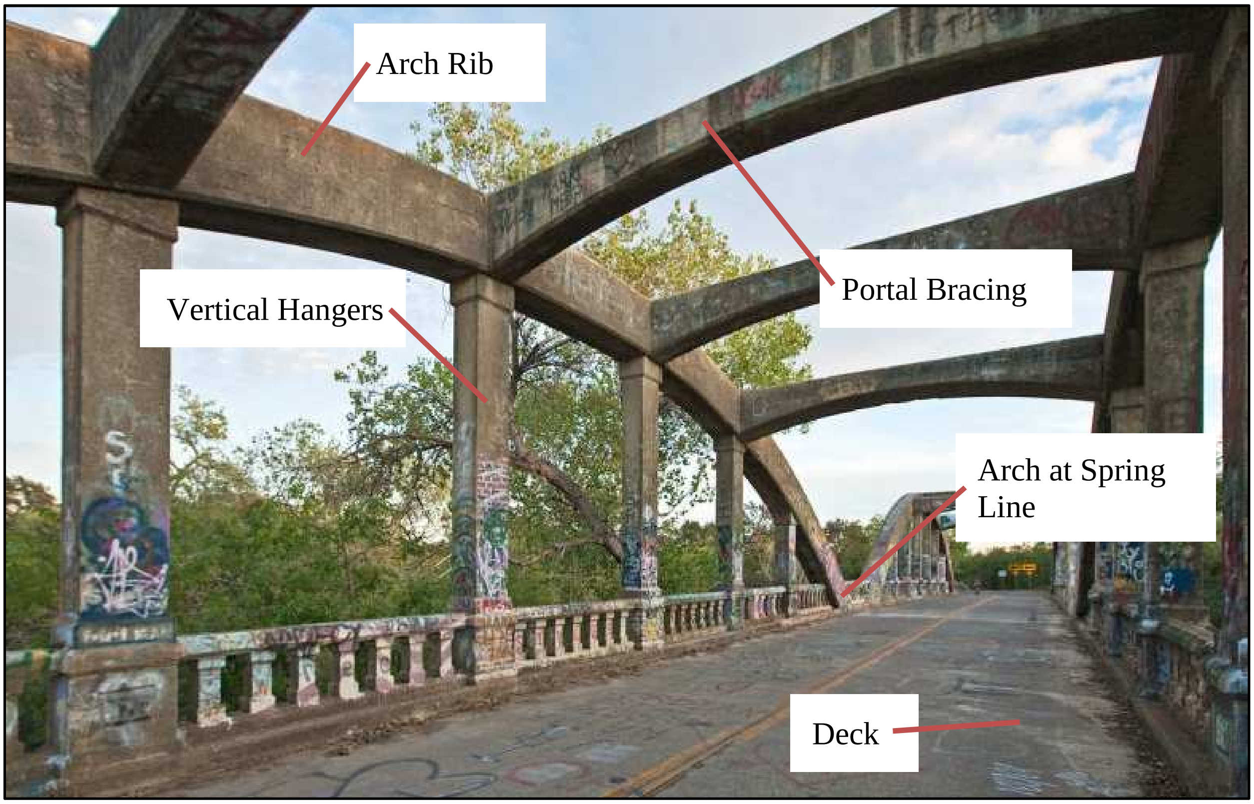


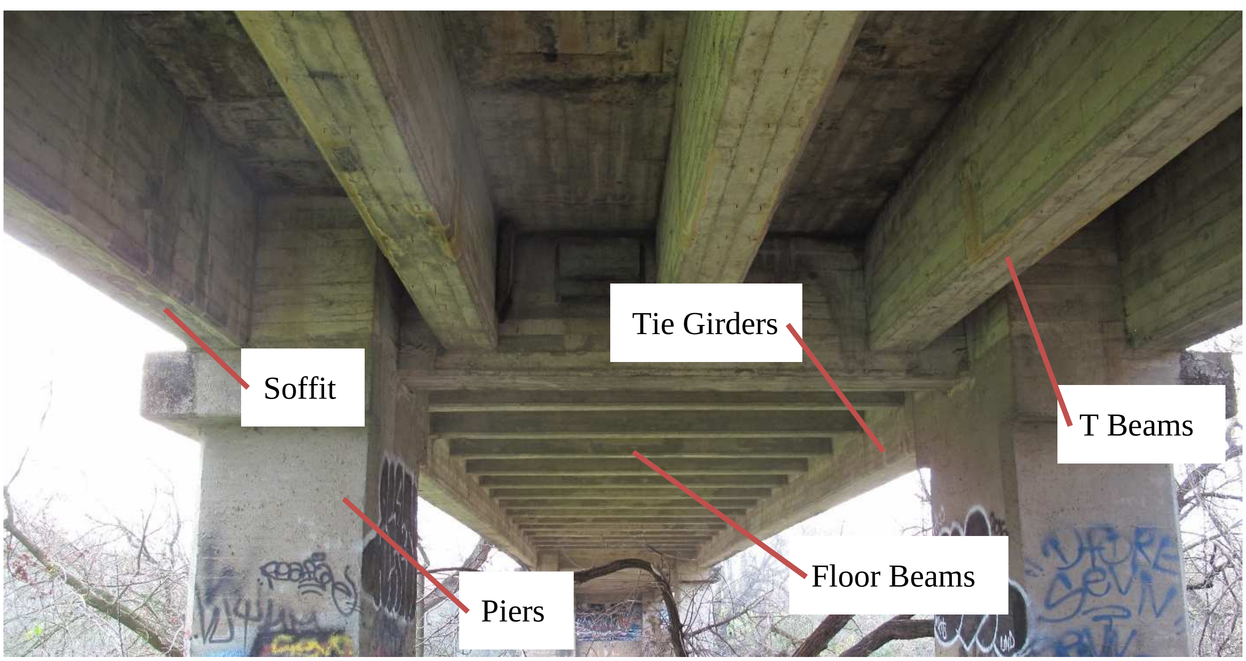
The Putah Creek Bridge, or the "Graffti Bridge" as it is known locally, has considerable public and historical interest. The bridge is one of three tied arch bridges in Northern California, and is considered historically significant. The same plans were used to construct the Rumsey Bridge located approximately 40 miles to the northwest. The Rumsey Bridge is currently scheduled to be replaced, which only increases the historical importance of the Stevenson Bridge.

Stevenson Bridge, standing at Abutment 1, looking north

Naming Convention

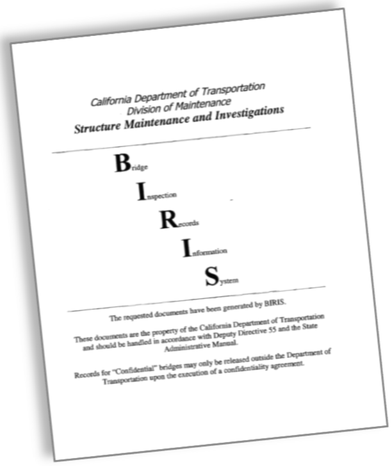
The bridge contains a number of structural components that are unique to this style of bridge. Past reports have referred to these elements using different names. For the purpose of maintaining consistency in this report, the naming convention for these structural elements is illustrated in the photos below.





Caltrans Bridge Inspection Reports

Over the years, Caltrans has completed evaluations of the bridge and produced Bridge Inspect Reports on a regular basis. The earliest inspection report within the Bridge Inspection Records Information System (BIRIS) system was prepared in 1971. Based on that report, soundings around Pier 3 indicated that approximately 3 feet of scour had already occurred around the existing piles. The report also cites numerous transverse deck slab cracks extending into the tie girders which are still present today.


California Department of Transportation
Division of Maintenance
Structure Maintenance and Investigations

Bridge
Inspection
Records
Information
System

The requested documents have been generated by BIRIS.

These documents are the property of the California Department of Transportation and should be handled in accordance with Deputy Directive 55 and the State Administrative Manual.

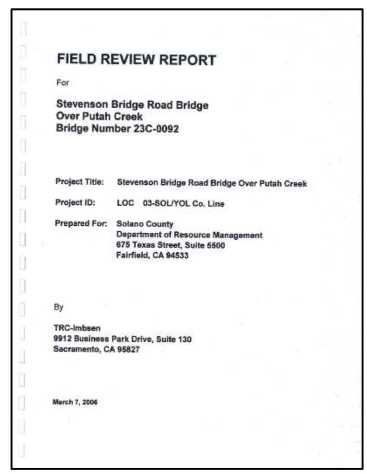
Records for "Confidential" bridges may only be released outside the Department of Transportation upon the execution of a confidentiality agreement.

Caltrans Maintenance Report

The latest bridge inspection report (March 25, 2015) classifies the Stevenson Bridge as being Functionally Obsolete. The Functionally Obsolete status is based on the existing deck geometry and approach roadway alignment relative to existing standards. The Caltrans inspection report notes cracks in girders at Spans 1 and 4 extending into the soffit. Additional cracks on girders at spans 2 and 3 are estimated at 20% of the length of the girders. The report also notes that transverse cracks at spans 1 and 4 appear to not have changed since 2009. Caltrans has made numerous work recommendations such as patching spalls and cleaning/painting exposed rebar to prevent further deterioration. The report also references the failure of the retaining wall at Pier 1. Scour and degradation also appear to be progressing. New scour measurements were compared with measurements taken in 2007 which indicated an additional 8 inches of degradation in the channel at Pier 3, and an additional 10 inches of degradation at Pier 4.

Past Studies

In 2006, the County contracted with TRC/Imbsen to perform a field review and make recommendations for possible repairs. TRC/Imbsen completed the Field Review Report in March of 2006. This report was subsequently followed by a feasibility study also prepared by TRC/Imbsen which was submitted in February of 2007. TRC/Imbsen presented two rehabilitation options, and a replacement option to assist the County in making a decision of how to proceed with the project.


FIELD REVIEW REPORT

For

Stevenson Bridge Road Bridge
Over Putah Creek
Bridge Number 23C-0092

Project Title: Stevenson Bridge Road Bridge Over Putah Creek

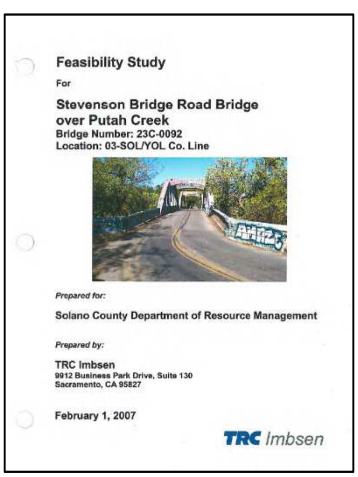
Project ID: LOC 03-SOL/YOL Co. Line

Prepared For: Solano County
Department of Resource Management
675 Texas Street, Suite 5500
Fairfield, CA 94533

By

TRC-Imbsen
9912 Business Park Drive, Suite 130
Sacramento, CA 95827

March 7, 2006


Feasibility Study

For

Stevenson Bridge Road Bridge
over Putah Creek
Bridge Number: 23C-0092
Location: 03-SOL/YOL Co. Line

Prepared for:

Solano County Department of Resource Management

Prepared by:

TRC Imbsen
9912 Business Park Drive, Suite 130
Sacramento, CA 95827

February 1, 2007

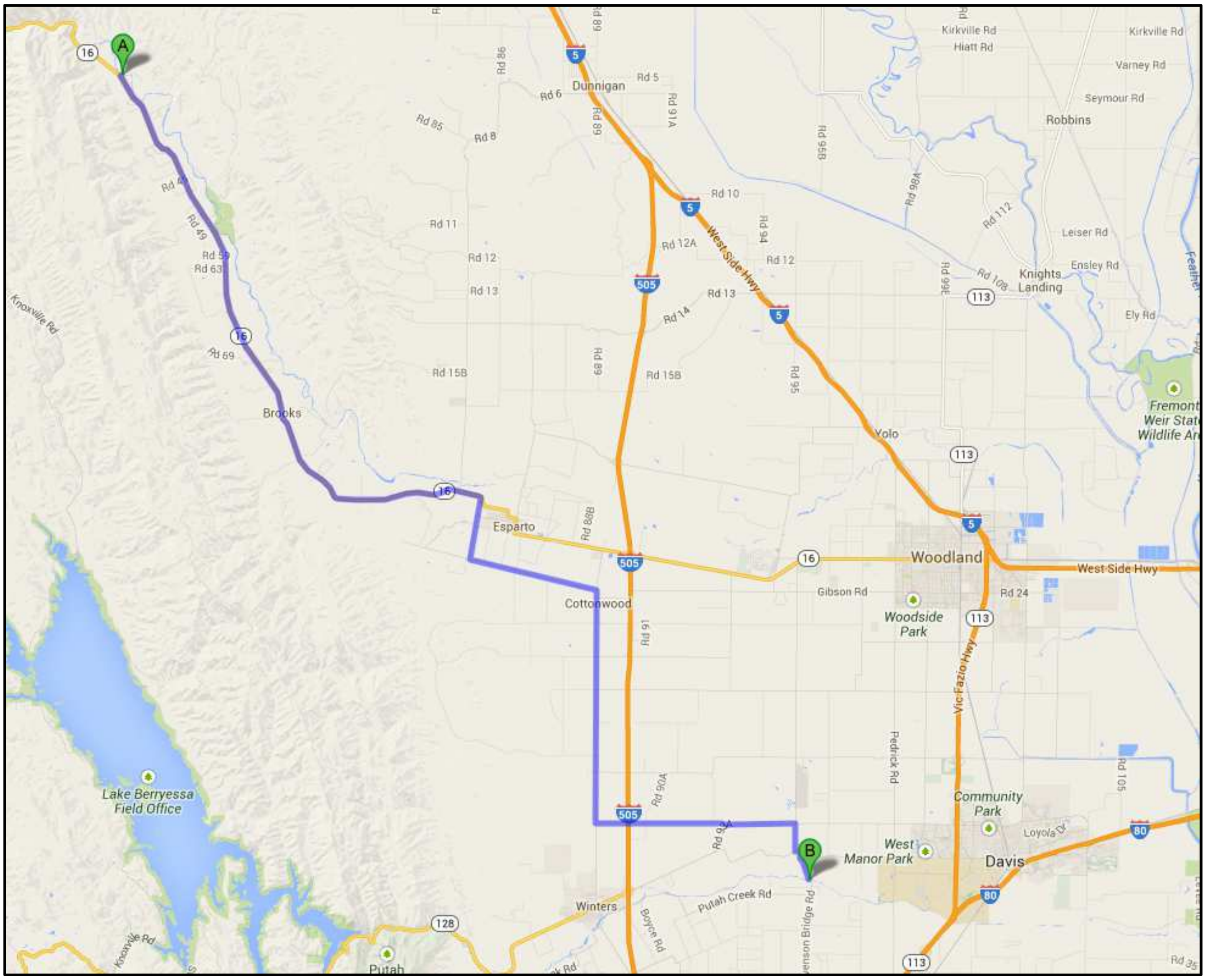
TRC Imbsen

Using information presented in the feasibility study, the County has elected to rehabilitate the bridge, and has completed the environmental process for this option. The County utilized Sycamore Environmental Consultants, Inc. and Mead and Hunt, Inc. to produce technical studies to support environmental clearance.

In February of 2016 the County solicited a Request for Proposals to produce plans, specifications, and estimate for the rehabilitation alternative. Quincy Engineering, Inc. was selected and began work in April of 2016.

Similar Structure (Rumsey Bridge)

There is another bridge in the general area that is nearly identical to the Stevenson Bridge. The Rumsey Bridge (22C0003) over Cache Creek is located only approximately 40 miles away. The Map below shows the Rumsey Bridge at location "A" and the Stevenson Bridge at location "B".



Google Map of Rumsey Bridge at Location A and the similar structure Stevenson Bridge at Location B

Quincy performed an assessment of the Rumsey Bridge for Yolo County under a separate contract in December of 2015. The report concluded that rehabilitation of that structure was much less feasible than a replacement. Since the main superstructure arch spans of the Rumsey Bridge are nearly identical to that of the Stevenson Bridge it is important to note why a rehabilitation is more feasible at this site.

While both bridges did share similar vulnerabilities, it is more feasible to rehabilitate the Stevenson Bridge due to the following reasons:

- The Stevenson Bridge is in overall better condition relative to the Rumsey Bridge. Some locations on the Rumsey Bridge, such as the Tie-Girder Soffit, are in such a deteriorated state that the surface concrete has spalled off over almost the full length and width of the elements.
- Portal Bracing serves as the primary lateral force resisting element when the bridge experiences transverse loads from wind or seismic events. The Stevenson Bridge has two additional portal braces

relative to the Rumsey Bridge. This means the Stevenson Bridge has more lateral resistance and is more seismically resilient to lateral loads than the Rumsey Bridge.

- While the bridges have a nearly identical design, they were constructed at very different sites. The Acceleration Response Spectrum (ARS) curve at the Rumsey Bridge site is higher than the ARS curve at the Stevenson Bridge site. This means lower seismic demands will be imposed on the Stevenson Bridge during an earthquake.
- The Stevenson Bridge is supported on taller piers, which lead to a longer structural period of vibration. This means the bridge is more flexible and as such will attract lower seismic forces in the bridge.

In summary, the Stevenson Bridge is in better condition and has some site specific characteristics such as lower seismic effects and taller piers that make it a superior candidate for bridge rehabilitation/retrofit. While it is also feasible to retrofit the Rumsey Bridge, the cost would be significantly higher than the Stevenson Bridge because of its poor condition, site specific issues (scour, seismic demands) and its shorter, stiffer piers.

Historical Bridge Consideration

Under the National Bridge Inventory, this structure has a Historical Bridge Inventory Category Rating of 2, meaning this bridge is eligible for the National Register of Historic Places. Mead and Hunt, Inc. was retained to develop the Finding of Effect document which is required when historical resources are involved. The document concluded with a finding of no adverse effect. Therefore, the rehabilitation project will not adversely affect the historic resource as long as the PS&E can be completed within the parameter stipulated in the Finding of Effect document. Consequently, repairs must be in-kind or minimize changes to the member shapes and sizes visible to the public. Based on our analysis, the proposed repairs are consistent with those assumed for the Finding of Effect document. Therefore the proposed retrofit will not adversely affect this historic resource.

Stevenson Bridge

Need and Purpose

The Stevenson Bridge was originally constructed in 1923, and is classified by Caltrans as Functionally Obsolete per the latest maintenance report issued on 3/25/15 (Caltrans has recently eliminated this designation for all bridges). The structure has a sufficiency rating of 60.4 out of 100. While the existing tied-arch bridge has provided a functional creek crossing for the last 94 years, rehabilitation is necessary to restore the bridge to the As-built condition. Retrofit is also necessary to strengthen several members to reduce seismic and scour vulnerabilities.

Necessary repairs include spalled or delaminated concrete and exposed reinforcing steel. Flexural cracks present in both approach spans, and failure of the retaining wall at Pier 2 must also be addressed. Deficient components include the bridge railing, deck spalling, deck carbonation, deck drains, cracking of approach spans, and nonstandard south approach roadway alignment.

In addition to general repairs, the hydraulic analysis determined the structure is vulnerable to scour. Scour mitigation will be required to maintain the structural integrity of the bridge. At Pier 3, the scour depth is estimated to be approximately 26 feet while the as-built plans indicate that the existing timber piles at Pier 3 are only 40± feet long. Calculated scour combined with future degradation, which has been observed at this site since the early 1970's, could further lower the creek bed around the pier and pose a significant threat to its foundation. If the calculated scour were to occur, the timber piles would have only 14 feet of embedment at most. Under this condition, the piles would be unstable which could result in a collapse of this support and possibly both arch spans. Due to exposed piles, and the lack of adequate scour protection at Pier 3, the Stevenson Bridge is defined as “scour critical.” This means that one or more of its support are vulnerable to scour attack that could lead to the loss of support at one or more locations and the potential for a partial or total collapse of the bridge.

Not only is the existing structure hydraulically vulnerable, it is also susceptible to collapse during a seismic event. Seismic assessment of the bridge showed that many of the existing structural components of the bridge are unable to withstand seismic loads without retrofitting. Flexural and shear demands exceed the corresponding capacities. Due to the massive weight of the tied-arch superstructure and stiffer pier wall substructure, the bridge attracts very large forces during an earthquake. These forces, combined with poor structural details in the superstructure (Arch Ribs, Vertical Hangers, and Tie-Girders), make the bridge susceptible to collapse during a significant seismic event. See the Existing Bridge Seismic Assessment (As-built Model) section of this report for more information on the seismic vulnerability of the bridge.

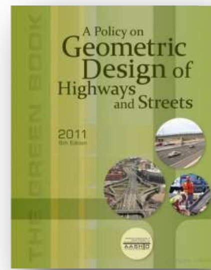
For the existing bridge to remain serviceable and safe into the future, repairs, seismic retrofitting, and foundation enhancements are necessary to make the structure resistant to extreme events like large storms and earthquakes. The proposed project will improve public safety by providing a safe creek crossing, and allow for this historic resource to remain in place for future generations to enjoy.

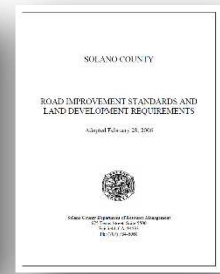
2. DESIGN AND CONSTRUCTION CONSIDERATIONS

Design Criteria

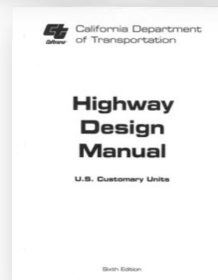
- Roadway Design

Several documents were used to determine the project design criteria including:





- Solano County Road Improvement Standards and Land Development Requirements (County Standard) dated 2006,
- AASHTO's "*A Policy on Geometric Design of Highways and Streets*", 2011 Edition,
- Caltrans "*Highway Design Manual*".



Where there are discrepancies between the design documents, the AASHTO standards shall be used as long as they do not worsen the existing condition. The summary of the project minimum design criteria are as follows:

Functional Classification - Major Collector in Level terrain

ADT (from 2015 BIRIS) - 789 (Year 2008); 1518 (Year 2035)

Proposed Design Speed - 35 mph

In accordance with the requirements stated in AASHTO's "*A Policy on Geometric Design of Highways and Streets*", 2011 Edition, the appropriate design speed for a major collector in level terrain is 50 miles per hour for design volume between 400 and 2000 vehicles per day. The County standard defers to AASHTO design speed requirements with several exceptions which are not valid for this project so the AASHTO standard will control. In order to reduce right of way impacts, the County will utilize a 35 mph design speed which will require a design exception. The structural section will be 0.45' of Asphalt over 1.70' of Class 2 AB. This section is based on a TI of 9.

Maximum superelevation rate - 6% (emax=6%)

Grades - 0.5% min to 7% max

Lane Widths - 24 foot traveled way based on County Standards (pg. 5, Sec. 1-2.7)

Shoulder Widths - 4 foot paved shoulders based on County Standards (pg. 5, Sec. 1-2.7). Note that since the bridge is only 24' wide and will not be widened this standard applies to the roadway approaches only.

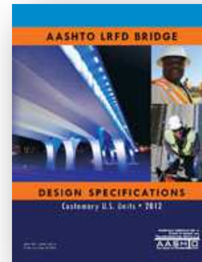
Cut Slopes - 2:1 (h:v)

Fill Slopes - 2:1 (h:v)

See the Design Criteria Memorandum in **Appendix C** for a comparison between County and AASHTO standards.

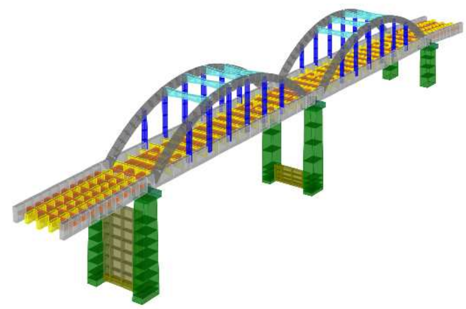
- Bridge Design

  Final bridge design will be performed in accordance with *AASHTO LRFD Bridge Design Specifications, Sixth Edition, and the latest Caltrans Amendments* (current version is AASTHO-CA BDS-6 with Caltrans amendments dated 2014). The latest updated versions of Caltrans bridge design manuals such as Memo to Designers 20-4 will also be utilized when applicable.



- Seismic Design

  For <u>rehabilitation/retrofit</u> of the existing bridge, seismic assessment and design will be based on a no-collapse criterion. A 3-dimentional finite-element global model will be created to assess seismic force, displacement, and rotation demands. Local nonlinear moment-curvature models for each non-elastic element type will be used to determine local member forces, displacement, and rotation capacities. See Existing Bridge Seismic Assessment (As-built Model) of this report for an in-depth retrofit methodology. Where applicable, methodology of the latest Caltrans Seismic Design Criteria (SDC) (current version is Version 1.7 dated April 2013) will be used. Note that the SDC criteria is intended for new construction only and is not a required criteria for retrofit design.



Global 3D Model

Traffic/Detour

The proposed retrofit and repairs will require the bridge to be closed during construction. With an average daily travel of 789 vehicles per day, this bridge provides a vital link between Solano and Yolo Counties for emergency vehicles, fire protection, and residents. This bridge also provides a reduced traffic alternative for bicycle riders traveling between Davis and Winters. Without the bridge to provide access, a detour will be required during construction. Available detours are approximately 15 miles in length to the west, and 10 miles in length to the east. The western detour utilizes the State Route 505 crossing while the eastern detour utilizes the Pedrick Road crossing. The Pedrick Road detour may only be feasible for north and south vehicles traveling to and from Yolo County from Interstate 80. Only narrow farm roads exist between Pedrick and Putah Creek Road south of the bridge which would not be able to accommodate a higher level of traffic. Therefore, it may be preferred to make the official detour State Route 505 to the west.

Bridge Railings

For local agency projects to qualify for federal funding, Caltrans Structures Local Assistance indicates that new bridge railings must conform to the full-scale crash-test criteria established in *Manual for Assessing Safety Hardware* (MASH) and *National Cooperative Highway Research Program* (NCHRP 350). For a 35 mph design speed, an appropriate railing should satisfy TL-2 crash test requirements or greater.

For rehabilitation projects, the existing bridge railing can be repaired or replaced in kind. Preservation of the existing rail is preferred, and replacement of the rails will only be considered if repair is unfeasible. Due to historical considerations for this structure, the exterior rail appearance must replicate the appearance of the original rail. Since this rail has not be crashed tested a design exception may be necessary to replace in kind.

Approach Guardrail

Approach guard railing is typically required at bridge crossings to protect oncoming traffic from the blunt end of the concrete bridge rail. If the bridge is wide enough, guard railing can be omitted on the departure side of the bridge. Based on the design speed and clear width between the concrete rails at this site, guard railing and protective end treatments are required at all four corners of the bridge.

The standard approach guard railing should meet FHWA's MASH and NCHRP 350 requirements, which would include a 25' long stiffened section of Caltrans standard Midwest Guardrail System Transition Railing (Type WB-31) adjoining either a flared (37.5' long) or in-line (50' long) terminal system. This guardrail application is feasible on the north side of the bridge where the roadway approach is straight, however this rail may not be feasible on the southern approach due to the curved alignment. These rails have only been crash tested on a tangent alignment so a curved application may require a design exception or not be feasible. Since guard rails would change the appearance of the bridge they may not be feasible, since they could result in an adverse impact to the historic resource. Further coordination with the County, a review of the accident history and a review of the Finding of No Adverse Effect document is necessary to determine if it is prudent to install approach guard railing.

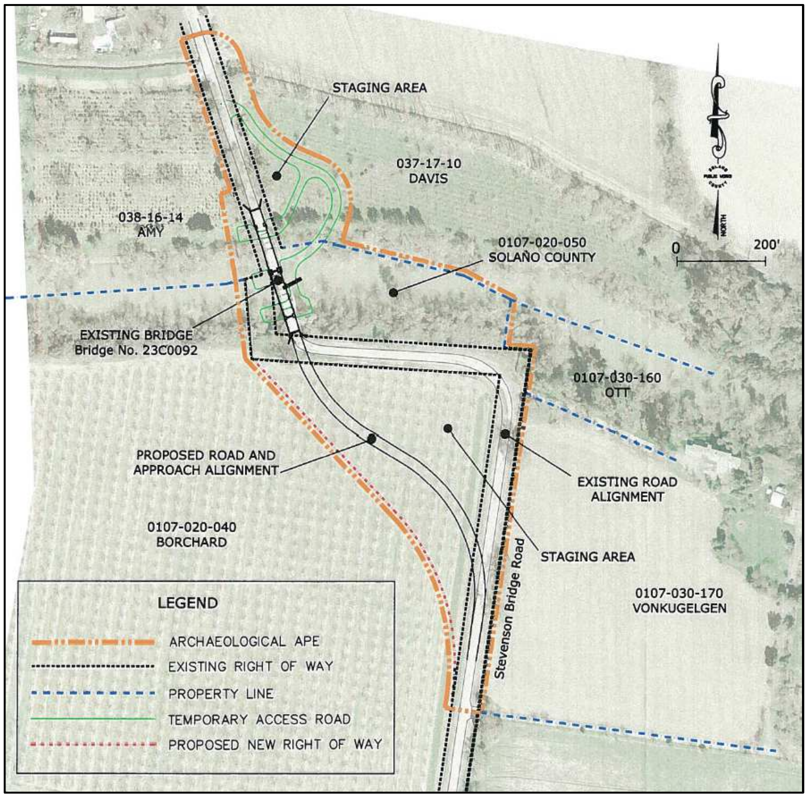
Design Exceptions

While the design standards are a mixture of AASHTO and County Standards **(Appendix C)**, it is important to note that the rehabilitation will match or improve the existing condition. Possible design exceptions for standards are summarized below:

- Roadway Geometry
- Bridge Railing
- Approach Guard Railing
- Bridge Clear Width

Contractor Access

Contractor access to install retrofit piles and install rock slope protection will be achieved via temporary roads cut into the north and south banks on the east side of the existing bridge. These access roads have been environmentally cleared and are necessary to allow large equipment such as cranes, drill rigs, excavators, dozers, and dump trucks to enter the steep creek channel to perform rehabilitation construction. It is anticipated that equipment will travel under the bridge to access retrofit locations to the west, however portions of the crane leads may need to be disassembled to allow enough vertical clearance. It is anticipated that the contractor could suspend temporary work platforms from the existing structure to access other retrofit locations under the bridge. Scaffolding or falsework supported from the ground under the bridge may also be necessary. Since the bridge will be closed to traffic, man lifts could be used on the existing bridge deck to access other retrofit/repair locations above or adjacent to the structure.



Staging Areas

Contractor staging areas are proposed in the southeast quadrant and have been shown on the area of potential effect map. While the existing road will be closed during construction it is anticipated areas outside the County right of way will be necessary to provide the contractor with sufficient room to construct the project. A temporary construction easement will be procured for these areas during the right of way phase. Environmental restrictions typically prevent the storage of materials and equipment within the creek banks which is why a large flat area outside the creek limits will be necessary for staging.

Right of Way

The existing County Road right of way is 60' wide but is not always centered on the road. Most of the southern portion of the bridge is actually located inside of a separate parcel owned by the County (APN 0107-020-050). All bridge rehabilitation improvements will occur inside existing County right of way. Additional permanent right of way would be necessary in order to improve the southern roadway approach. Temporary Construction Easements (TCE) are also anticipated to be necessary to provide adequate room for construction, provide room for temporary access roads and for contractor's staging areas.

Community and Cultural Outreach

In December 2010, the Native American Heritage Commission (NAHC) was contacted with a request for a query of their Sacred Lands File and a list of Native American contacts. The NAHC responded in December of 2010, noting that no Native American cultural resources had been recorded within the project area. The NAHC also provided a list of Native American individuals and organizations that might have concerns with or interest in the current undertaking. Native American individuals and organizations were contacted by letter in January of 2011. These included Kesner Flores, the Cortina Band of Indians, Dave Jones of the Wintun Environmental Protection Agency, and several individuals from the Yocha Dehe Wintun Nation: Marshall McKay, Leland Kinter, Cynthia Clarke, and Reno Franklin. Follow-up phone calls were conducted on June 9, 2011. One letter was received from Marshall McKay, dated January 11, 2011, stating that while their Cultural Resources Department has not identified any known sites within the project area, the project is situated within the aboriginal territories of the Yocha Dehe Wintun Nation. They requested a site visit to evaluate their concerns and determine the best management course. The field visit occurred on March 31, 2011. Additonal information regarding the results of Native American coordination, and public involvement related to archaeological resources are provided in the HPSR.

In an effort to establish public outreach and to inquire about the local history of the project area, relevant preservation groups within Solano and Yolo County, including the Solano County Genealogical Society, Solano County Historical Society, Yolo County Historical Museum (Gibson House), and the Yolo County Historical Society, were contacted in January 2011. No responses were received during these efforts.

In addition, a meeting with the Lower Putah Creek Coordinating Council to present the proposed Stevenson Bridge Seismic Retrofit Project was conducted in Vacaville, California with the public in December of 2013. Public comments on this proposed project were addressed by Solano County at this public meeting administered by the Lower Putah Creek Coordinating Council. Copies of this public involvement correspondence are included in the HPSR.

After the August 2014 seismic event in Napa County, American Canyon, and surrounding areas in Solano County, there was an immediate rise in interest from the public on the status of the rehabilitation of the Bridge. Concerns were related to any potential damage the bridge may have suffered from the event, as well as its exposure to future events. It should be noted that the local farmers and cyclists are particularly interested in this project as it provides a vital link between the two counties across Putah Creek. The County will include discussion regarding the seismic inadequacy of the bridge in all planned future public meetings at the Board of Supervisors (Solano and Yolo Counties), Solano County Water Agency, and the Lower Putah Creek Coordinating Council.

In April of 2015 as part of public participation under Section 106, the County sent letters to the Solano County Historical Society, Solano County Genealogical Society, Yolo County Historical Museum (Gibson House), Yolo County Historical Society, Historic Bridge Foundation, and the California Preservation Foundation. These letters described the proposed project and asked for comments in reviewing the Finding Of Effect (FOE) and Secretary Of The Interior's Standards (SOIS) Action Plan documents. To date, the County has not received any responses. Copies of correspondence related to the public participation are located in the Finding of Effect document for the proposed project.

Utilities

The following utilities were observed at the site:

- PG&E overhead electric (supported by independent poles West of the bridge)
- AT&T overhead telecom (supported by independent poles East of the bridge)

While no utilities are supported by the existing structure, relocations may still be required to provide adequate clearance to overhead lines adjacent to the bridge. Large diameter CIDH retrofit piles which are proposed at all supports locations require high overhead clearance for cranes during installation. PG&E requires construction buffers to their lines which can change based on the voltage but are typically not less than 10'.

Environmental

The design of the proposed project will minimize environmental impacts as much as possible. Environmental studies have been completed in compliance with federal and state requirements for National Environmental Protection Act (NEPA) and California Environmental Quality Act (CEQA). Solano County was the lead agency for the completed Initial Study/Mitigated Negative Declaration CEQA document. Caltrans was the lead agency for the completed Categorical Exclusion NEPA document. The following technical studies were completed and approved to support the environmental documents:

- Area of Potential Effect Map (APE)

- Historic Property Survey Report/Archeological Survey Report (HPSR/ASR)
- Finding of Effect (FOE)
- Historic Resource Evaluation Report (HRER)
- Natural Environment Study (NES)
- Biological Assessment (BA)
- Biological Opinion (BO)

Falsework

Falsework requirements at the site may vary from falsework placed in the creek to support construction of the interior Tie-Girder bolster, to falsework or work platforms suspended from the existing bridge for rehabilitation/retrofit access. While Elderberry bushes exist at the site, they are not located under the bridge. Therefore, there are no known environmental restrictions or mitigation measures on this project that would preclude the use of falsework in the creek; however creek flows may need to be considered during the falsework design. The allowable time the falsework can remain in the Creek may also be subject to creek flows and environmental permit requirements. Based on likely permitting requirements the construction window for work in the creek is restricted to between June 1st and October 15st. This 4.5-month window should provide an adequate amount of time to construct the retrofit/rehabilitation.

Temperature

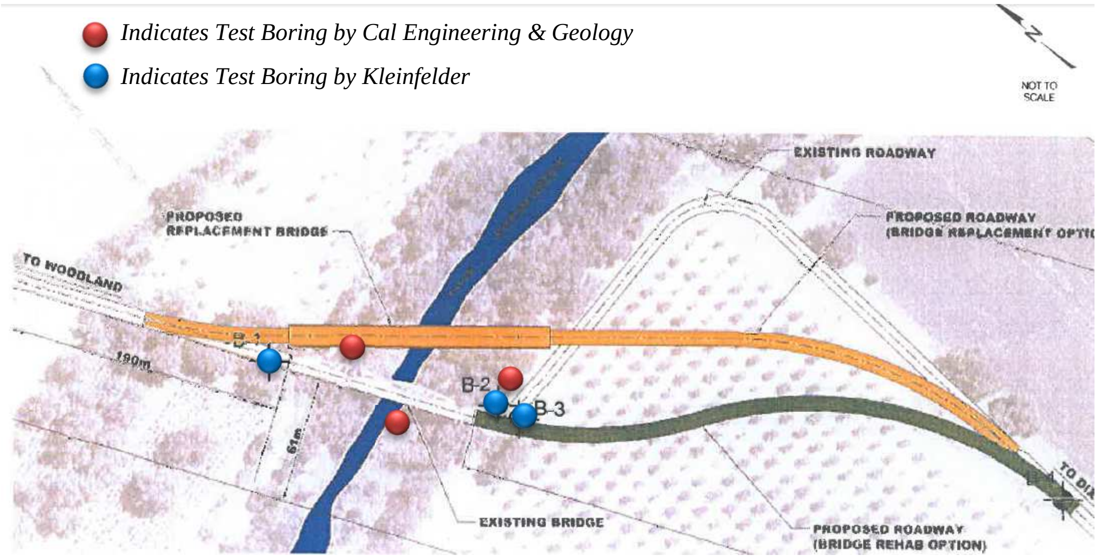
Maximum Temperature: 115° F
Minimum Temperature: 15° F
Obtained from weather.com record temperatures for Winters CA

Deck Protection and Corrosion

The project is located in a non freeze-thaw climate based on Caltrans Memo to Designers. Based on geotechnical borings corrosive soils are not present. Based on these conditions special details such as increased concrete cover or epoxy coated reinforcement will not be required.

3. GEOTECHNICAL

A draft foundation report has been prepared by Cal Engineering & Geology which is located in **Appendix E**. Recommendations presented in the report are based on three test borings obtained between September 12th and October 20th, 2016. These borings are supplemented by data from Kleinfelder obtained from three test borings drilled in December of 2005 shown below.



The project site is situated within the Great Valley Geomorphic Province near the western boundary. This portion of Solano and Yolo Counties is comprised of primarily marine and non-marine sediments deposited within the late Cenozoic Era. Material primarily consists of gravels, sands and silts.

Large Diameter Drilled Shaft example

Both cast-in-drilled hole (CIDH) concrete piles or driven piles could potentially be used at this site. Small diameter pile footings are less feasible as they would be less economical compared to large diameter CIDH or CISS concrete piles. Pile footings require a larger construction footprint, and would be less able to withstand the larger scour values present at this site. Large diameter CISS piles are feasible but would be more costly than large diameter CIDH piles.

The team recommends installing two large diameter CIDH concrete pile shafts adjacent to each existing pier footing. The large diameter piles will be tied to the existing footing cap by the means of installing a larger out-rigger footing that encapsulates the existing footing. Two large diameter CIDH concrete pile shafts are also proposed behind each abutment. These piles would be tied to the existing diaphragm with drill and bond dowels.

4. HYDRAULICS

A hydraulic report has been prepared by WRECO which is located in **Appendix F**. The existing channel is approximately 45 feet deep. Analysis shows that the existing structure provides adequate freeboard during the 50 year and 100 year storms to satisfy Caltrans criteria.

The primary hydraulic design considerations are the observed degradation and calculated abutment/pier scour. 2.5 feet thick Rock Slope Protection (300 lb, Class IV) (RSP) is proposed as a scour countermeasure at the bridge abutments. The RSP was designed using engineering judgement, the Caltrans "California Bank and Shore Rock Slope Protection Design" and FHWA's "Hydraulic Engineering Circular No. 23" (HEC-23). The code does not allow for RSP to be used as a scour countermeasure at the piers, therefore large diameter CIDH piles are proposed to keep the existing structure stable during large flow events. The calculated 25.7 feet of local pier scour would make the existing bridge unstable because as-built plans show that the existing timber piles are only 40 feet long. The proposed footing retrofit would prevent bridge collapse during the maximum scour event.

5. STRUCTURAL ASSESSMENT (PROPOSED GENERAL REPAIRS)

A detailed structural assessment was performed by Alta Vista in late 2016. A field investigation report is located in **Appendix G**. Alta Vista utilized visual inspection, borescope observations, ground penetrating radar (GPR) scanning and concrete core testing methods to assess the condition of the existing structure. This report plays a vital role in the rehabilitation design. Since the existing bridge is over 94 years old, repairs are required in order to restore the structure back to the existing as-built condition. The seismic assessment (as-built model) presented in the following sections of this report assumes that these repairs have been made so that as-built details can be used in the analysis with no reductions for damaged areas.

Repair recommendations primarily include removal of existing unsound concrete, cleaning and painting exposed reinforcing steel to prevent further deterioration, and patching spalls with new concrete. Methcrylate is also proposed for the deck to seal smaller cracks as well as epoxy injection for cracks larger than 0.01". Alta Vista documented each spall location and area in their report located in **Appendix G**. In summary Alta Vista recommends repairs for 2,258 sqft of concrete surface area. This was comprised of approximately 775 sqft of deck area, 40 sqft of girder area, 1,406 sqft of soffit area, and 37 sqft of arch, portal, and vertical hanger area. Volume of concrete spall repair are very difficult to estimate since the limits of unsound concrete removal can't be determined until the removal operation begins. In addition, it is also very difficult to estimate the quantity of epoxy crack injection because the size of the cracks can't be viewed for some members until the unsound concrete is removed. Quincy estimated the removal of unsound concrete volume based on a review of site photos and a depth assumption relative to the repair class specified in the Alta Vista report. For the epoxy crack injection Quincy assumed two full depth cracks per approach span.

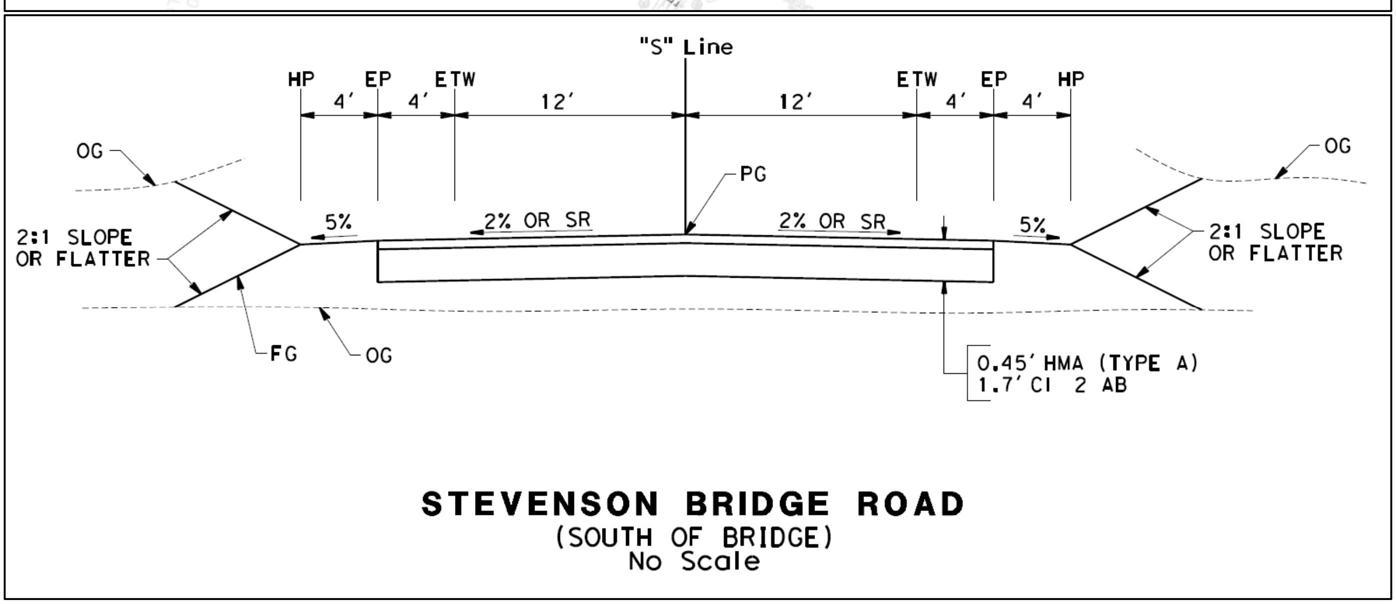
6. ROADWAY LAYOUT

As stated earlier the minimum design speed based on AASHTO standards is 55 mph. This high speed is not appropriate at the site given the existing narrow bridge and other right of way constraints. Currently the southern bridge approach has a horizontal curve radius of approximately 50' which if super elevated at 6% equates to a design speed of approximately 15 mph.

Other safety features such as standard guard rail and wider 4' paved shoulders with 4' graded aggregate base shoulders are proposed to improve safety relative to the existing condition. A design exception is appropriate as it would not be reasonable and prudent to increase the design speed to 55 mph at this site.

<u>35 mph alignment</u>

This alignment proposes to use 350' radius horizontal curves super elevated at 6% to achieve a design speed of 35 mph.



STEVENSON BRIDGE ROAD
(SOUTH OF BRIDGE)
No Scale

7. EXISTING BRIDGE SEISMIC ASSESSMENT (AS-BUILT MODEL)

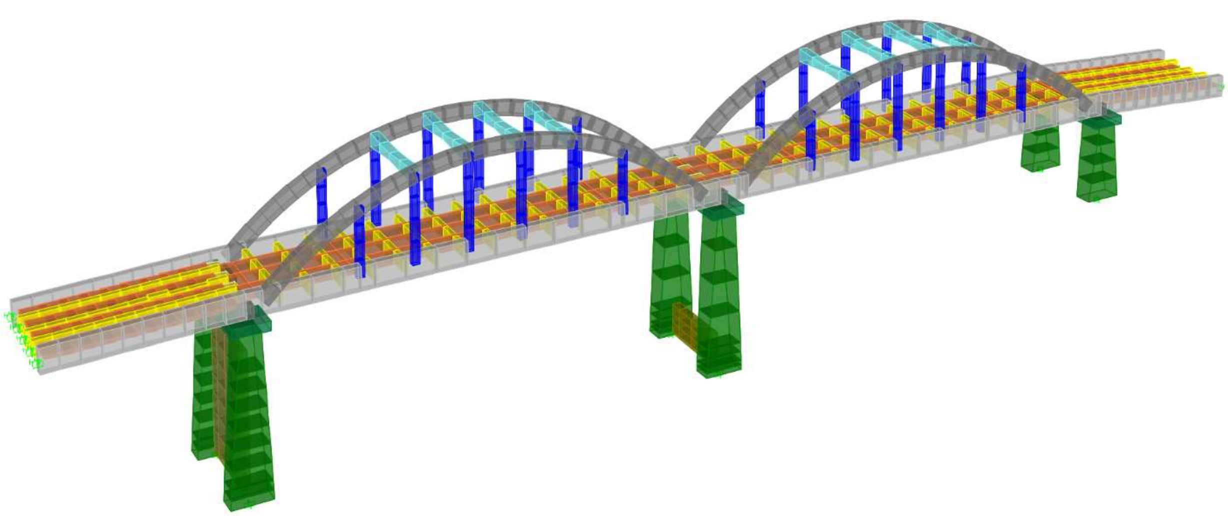
This section addresses seismic deficiencies only. Please refer to other sections of this report for general repairs such as concrete spalls, cracking, and scour deficiencies.

The following documents and information were used to support the seismic assessment:

- The 1923 Design/As-built plans
- All available Caltrans Bridge Inspection Reports
- Field Review Report (Imbsen/TRC March 2006)
- Geotechnical Investigation Report (Kleinfelder April 2006)
- Feasibility Study Report (Imbsen/TRC February 2007)
- Foundation Report (Cal Engineering & Geology October 2016)
- Field Investigation Report (Alta Vista January 2017)
- Design Hydraulic Study (WRECO January 2018)

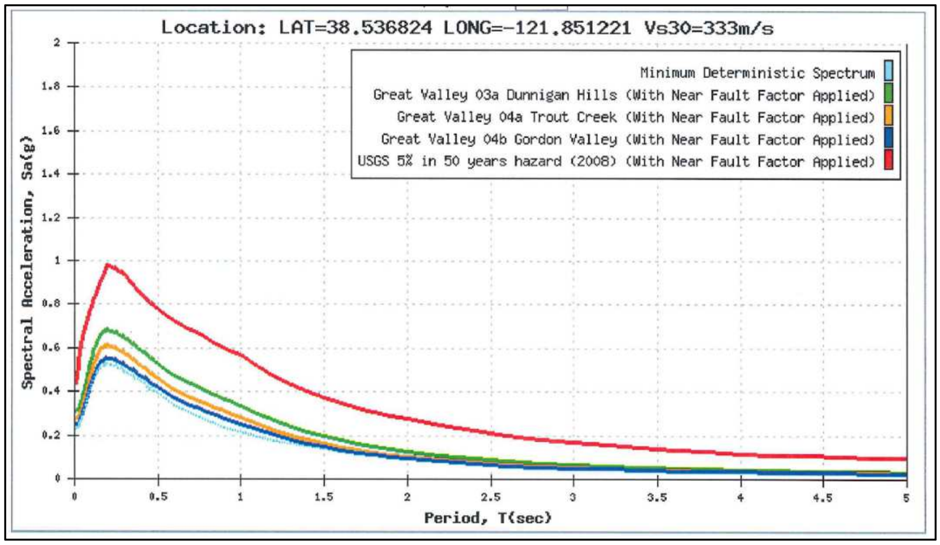
<u>Global Seismic Performance Criteria</u>

The Putah Creek Bridge has been evaluated to meet the performance requirement of "No-Collapse", which means that the bridge could be significantly damaged during an earthquake, but would not collapse. This conforms to the Caltrans design methodology (stipulated in Memo to Designers 20-4) and industry practice for bridge seismic design in California. After an earthquake, the bridge may require extensive repairs, or may have to be replaced entirely, but it would remain standing through an earthquake to minimize the threat to public safety during the event.



Elevation View of the Existing Bridge As-Built Model

Analysis Methodology



The Putah Creek Bridge is considered to be a Non-Standard Bridge by the Caltrans *Seismic Design Criteria* due to its unique superstructure type. To capture its complex seismic response, this bridge requires a more detailed analysis than typically prescribed. Therefore, an explicit elemental level dynamic analysis model was created to capture the effects on individual structural elements, including the Arch Ribs, Tie Girders, Vertical Hangers, Portal Bracing, Floor Beams, and Piers. This analysis was completed utilizing Structural Analysis Program Version 19 (SAP) created by Computers and Structures, Inc. A multimodal linear elastic dynamic analysis was performed with a Soil Type-D (Vs_{30}=333 m/s), controlling probabilistic acceleration response spectrum (ARS) curve increased 20% for periods greater than 1 second for near fault effects (project located less than 15 km from a fault plane), and a 5% damping ratio. 150 modes were required to obtain a mass participation ratio of more than 90%.

The structure was modeled explicitly with individual member properties (both gross and cracked inertia as discussed further below) along with boundary condition restraints, releases, and springs where applicable (also discussed in more detail below).

In order to confirm that the model was set up correctly, deadload reactions at the abutment and pier locations were obtained from the model. These results were compared to independent hand calculations of tributary dead loads based on the As-Built plans and member sizes. These two results correlated within 1% indicating that the length, orientation and areas of members described in the model match the As-built plans.

Material Properties

A concrete compressive strength (f_c') of 3,000 psi was used for the arch rib, vertical hanger, portal bracing, and pier elements. A concrete compressive strength (f_c') of 3,500 psi was used for the tie girder and deck elements in the baseline assessment model. These values were recommended by Alta Vista based on destructive testing results from concrete cores taken from the existing bridge and concrete strength information obtain by Kleinfelder during their structure assessment in 2006.

Concrete Core Compression Test Report

Client: Justin Chen
Business Name: Alta Vista Solutions
Address: 3260 Blume Drive, Suite 500
City/State/Zip: Richmond, California 94806

BSK Project No.: C16-523-60L
Sample ID No.: 16-936
Permit No.:
Report Date: 12/8/2016

Project: Stevenson Bridge
Project Address:
City, State:
Structure: Concrete Structure
Core Date:
Specified Strength, (psi):

Sample	Date Tested	Tested By	Dimensions: Average Diameter (in)	Dimensions: Average Length (in)	Area (in^2)	Ratio (L/D)	Correction Factor	Break Type	Maximum Load (lbs)	Compressive Strength (psi)
1B	12/08/16	R. Cortez	3.63	4.70	10.35	1.29	0.935	3	39,405	3,560
2	12/08/16	R. Cortez	3.63	7.48	10.35	2.06	1.000	3	29,625	2,860
3	12/08/16	R. Cortez	2.66	5.12	5.56	1.92	1.000	3	19,400	3,490
4	12/08/16	R. Cortez	2.66	5.02	5.56	1.89	1.000	2	20,400	3,670
									Average	3,400

The modulus of elasticity for concrete (Ec) was determined using SDC equation 3.2.6-1 based on the estimated concrete strength. Ec was assumed to be 3,122 ksi for the 3,000 psi concrete and 3,372 ksi for the 3,500 psi concrete. For inelastic behavior, a maximum concrete compressive strain of 0.002 was used which is recommended value in the SDC for unconfined concrete.

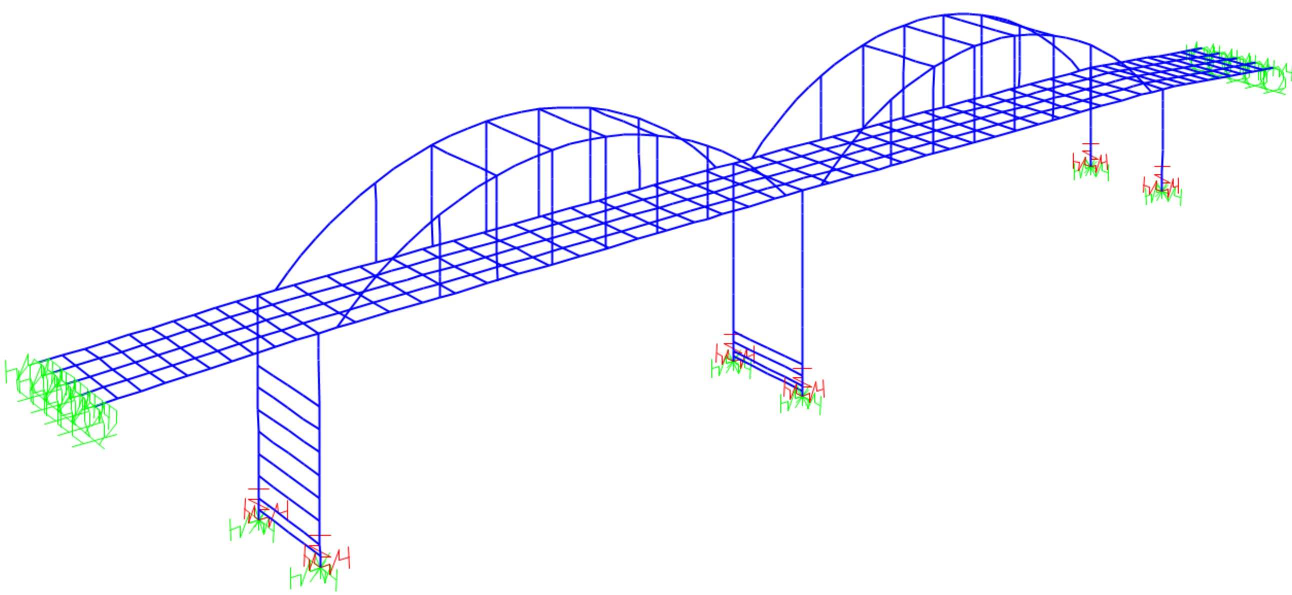
A yield strength (f_y) of 40 ksi was used for reinforcing steel based on historical material properties of rebar from the 1930's. The modulus of elasticity (Es) was assumed to be 29,000 ksi. For inelastic behavior, an ultimate strain of 0.06 was used for the reinforcement and is based on the uncertainty of the historical properties of reinforcement from the 1930s.

Scour Condition for Seismic Analysis

Since seismic is considered an extreme event per the LRFD load cases, the seismic analysis considered long-term degradation and contraction scour only. Local scour was neglected for the seismic analysis as prescribed by the code.

Boundary Conditions

Abutments for the As-built model were fixed for translation in the vertical direction. Springs were used for translations in both the longitudinal and transverse directions.



Longitudinal abutment springs were used to model the stiffness of the abutment-soil interaction. The spring force considered passive soil resistance along the back face of diaphragm per the method outlined in the Caltrans Seismic Design Criteria (SDC) section 7.8.1. Hand calculations showed that the abutment diaphragm did not have adequate structural capacity to engage passive soil resistance below the soffit or to engage frictional resistance along the bottom of the abutment spread footing. Therefore, the effective passive soil force area was limited to the portion of the diaphragm above the bottom of the girders. Since the passive force would only be mobilized in compression (when the superstructure moves towards the soil) the abutment spring force was divided by two because the springs work in both compression and tension and there is a longitudinal spring at each abutment. The longitudinal spring constants were iterated for force and displacement convergence while using gross section properties in the bridge elements.

Transverse abutment springs utilized the method outlined in SDC section 7.8.2. The existing wingwalls have a construction joint and minimal reinforcement so they are not capable of transmitting transverse superstructure loads into the soil. The SDC recommends that diaphragms abutments assume 40 kips/pile for the transverse spring force. Since these abutments are supported by spread footings, consideration was given to including the frictional resistance at the bottom of the footings in the spring force. Since the relatively thin diaphragm is likely to crack under longitudinal seismic loads it is not prudent to assume the diaphragm will be able to transfer the full transverse footing frictional resistance. Consequently, the transverse spring was set to a value equal to 50% of the adjacent pier stiffness which is the same approach used for seat abutments in the transverse direction per the SDC. All rotational degrees of freedom were released at the abutment locations, however vertical translation constraints at the bearings restrict rotations about the longitudinal axis.

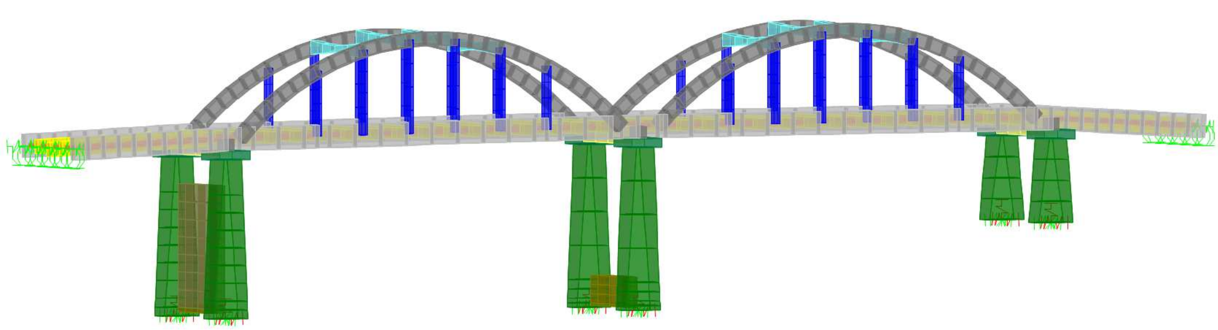
At the pier locations, the bottom of footing members were modeled as fixed for translation in the vertical direction. Longitudinal and transverse springs were iterated for force and displacement convergence to capture the behavior of the retrofit pile-soil interaction for translations in the longitudinal and transverse directions. Footing retrofits are required at all pier locations to address scour concerns regardless of seismic performance. For instance, the Pier 3 timber piles do not have sufficient embedment under the scour condition. Under this condition, the footing would be unstable and would have to be retrofitted just to maintain stability. Thus the As-built seismic assessment model did not consider a non retrofitted pier footing condition. All rotational degrees of freedom were released at the pier footing locations. Pier springs force and displacement convergence were set using gross member properties.

<u>Local Member Performance Criteria</u>

The primary collapse mechanisms for the Putah Creek tied-arch portion of the bridge would be the failure of the primary load carrying members, listed below:

<u>Superstructure Primary Members:</u>

- Arch Ribs (dark gray)
- Tie Girders (tan)

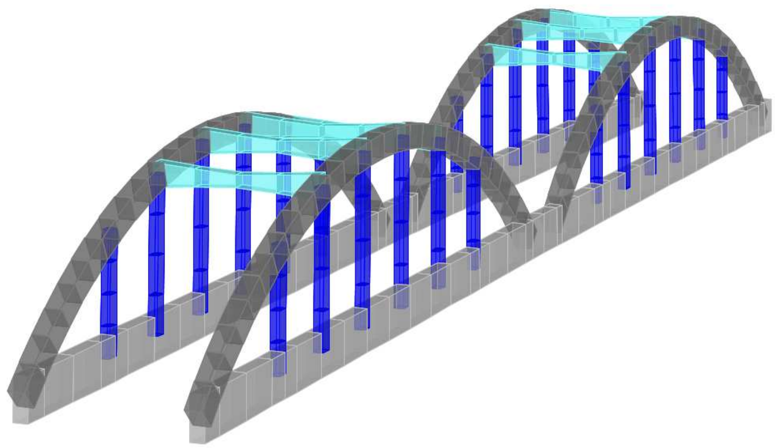


Since these members contain lap splices and a very limited amount of confinement reinforcement, the forces in these members should be limited to their yield capacity. In other words, inelastic, ductile behavior should not be permitted in primary members as failure occurs shortly after yield. The existing As-built plans also do not show any lap splices, however lap splices must exist based on the member lengths. Inelastic behavior beyond yield could result in loss of concrete cover which could affect the integrity of the lap splices. Ensuring that primary load carrying elements behave essentially elastic during a seismic event would thereby prevent the spans from collapsing. The primary member acceptance criterion for the As-built model is a force demand-to-capacity ratio of less than 1 (the demand on the member must be less than its capacity).

Secondary elements are also important to prevent structural collapse. However, secondary elements have more redundancy and could be allowed to behave inelastically. The following elements were considered to be secondary elements.

Superstructure Secondary Members:

- Portal Bracing (light blue)
- Vertical Hangers (dark blue)
- Transverse Floor Beams (not shown)



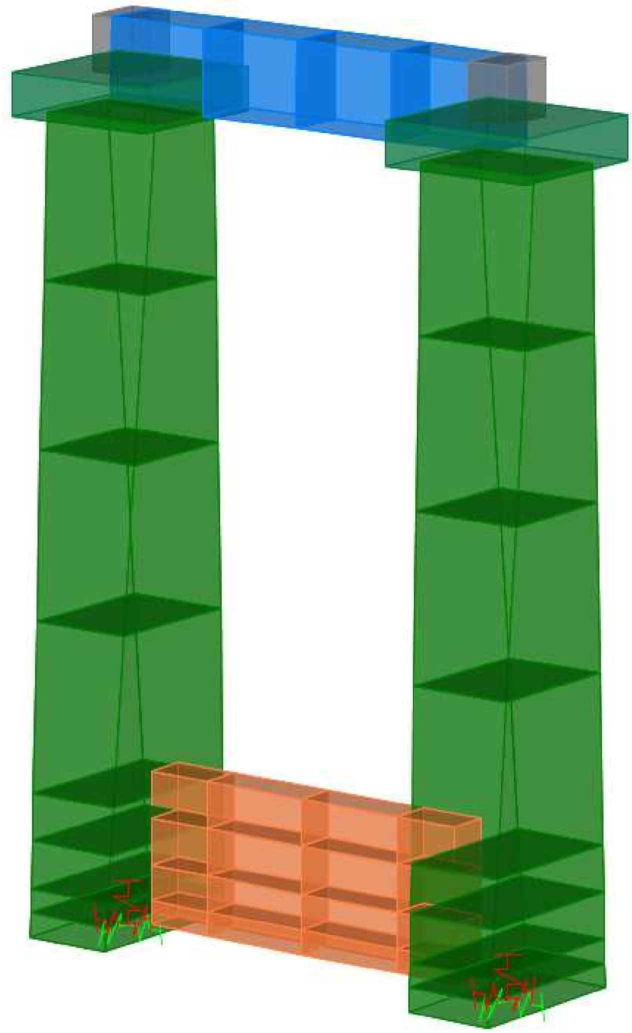
Flexural moment demand-to-capacity ratios could be greater than 1 for secondary elements, provided there is enough ductility and shear capacity to withstand the demands. Like the primary members, these secondary members also have a limited amount of confinement reinforcement. Local moment curvature models were run in SAP for these secondary elements. The analysis showed that they had very little ductility and failure could occur shortly after member yield. In addition to low ductility, secondary members were also deficient in shear. Consequently, it was determined that it would not be prudent to allow these members to behave inelastically in the As-built model. The secondary member acceptance criterion for the As-built model is a force demand-to-capacity ratio of less than 1 (the demand on the member must be less than its capacity).

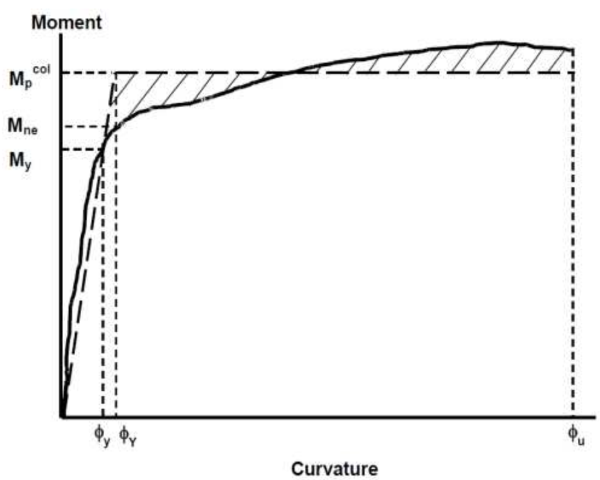
Based on the methodology outlined above, gross section properties were used for all superstructure structural elements to obtain force demands. If demands were found to exceed the yield capacity of an element, this indicated that the member would need to be retrofitted in order to prevent a possible collapse during an earthquake.

Substructure

- Pier Cap (blue)
- Piers (green)

Typical seismic methodology allows for piers/columns to behave inelastically during a seismic event provided there is enough ductility and shear capacity to withstand the demand displacements. Analysis showed that the moment demand of the piers was above the yield moment capacity therefore, moment-curvature analyses using SAP were performed on the existing piers to determine their inelastic behavior. This analysis considered the non-linear material behavioral characteristics of the concrete and rebar in the pier. This analysis was used to determine the strength and displacement/rotational capacities of the pier. Overturning axial effects (commonly called a push over analysis) were considered in the displacement capacity. The model was run a second time using cracked pier inertia in order to obtain the demand displacements.

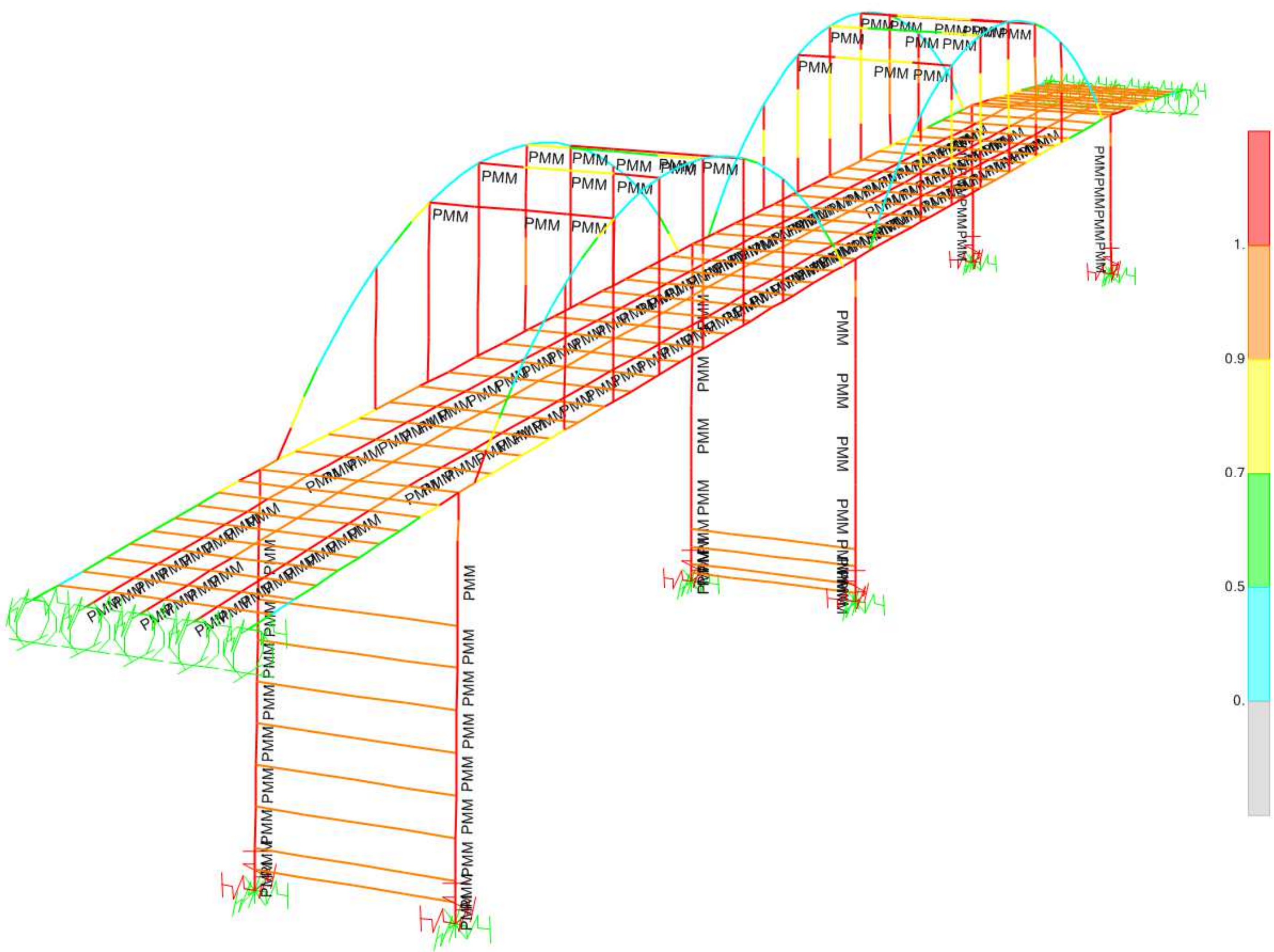




Moment-Curvature Analysis Curve

Existing Bridge Vulnerabilities

Once the capacities of existing members were determined and compared to demands from the dynamic analysis, it was determined that most superstructure elements do not have enough capacity to withstand seismic forces as shown in the demand-to-capacity (D/C) ratio color schematic below. A D/C ratio greater than 1 (shown below in red) indicates that the member demand has exceeded the member capacity.

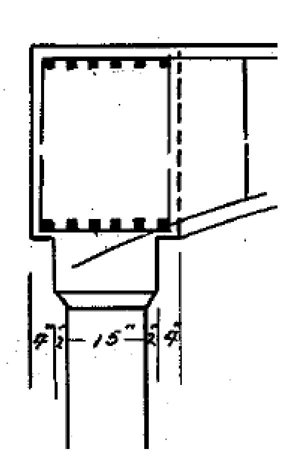


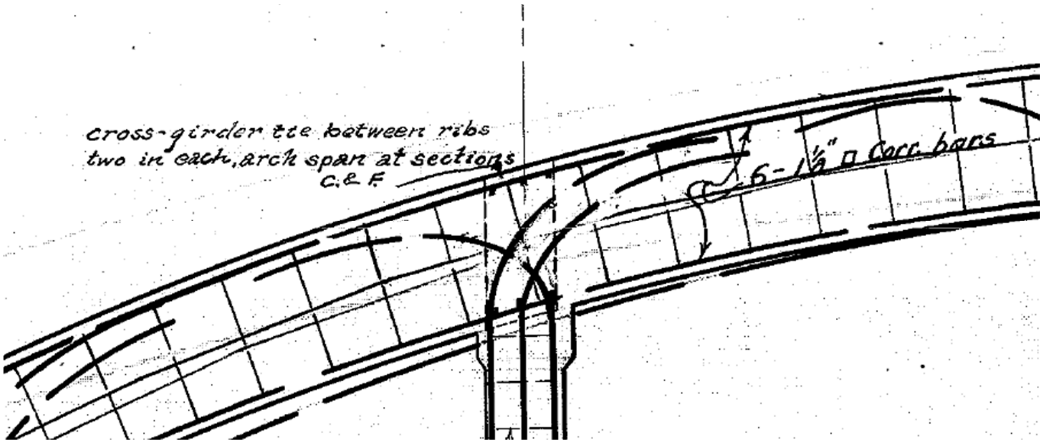
Demand-to-Capacity Ratios of Existing Bridge

The maximum demand-to-capacity ratios from the analysis are summarized in the following sections for each member:

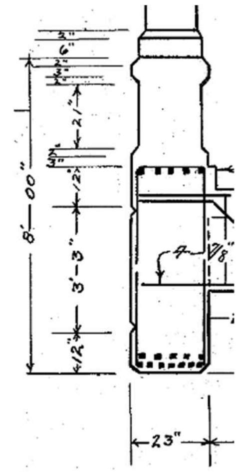
<u>Arch Ribs</u>

The existing arches are 36 inches deep by 27 inches wide. Per the As-built Plans and Ground Penetrating Radar (GPR) verification, the arch ribs are reinforced with six $1\frac{1}{8}$" square bars on the top and bottom faces. The $1\frac{1}{8}$" square main bars are roughly equivalent to the area of a current #10 round bar. For confinement, 3/8" square bars at 18 inch spacing are provided. The 3/8" square confinement bars are roughly equivalent to the area of a current #3 round bar. The existing amount of confinement is very minimal and does not meet today's minimum transverse reinforcement requirement. Without the minimum amount of transverse reinforcement, the arch is unable to restrain the growth of diagonal cracking and is unable to provide much ductility in a seismic event.





Under seismic loads, the arch members do not have sufficient capacity to meet the seismic bending and shear demands in both major and minor axes. The maximum moment demand occurs near the "spring line" adjacent supports of the arch. Based on the existing condition of the bridge, the maximum Demand-to-Capacity ratio of the Arch Rib elements for combined axial and flexural is 1.35. The shear strength comes mostly from the concrete since the shear reinforcement is minimal. The shear D/C = 0.70.



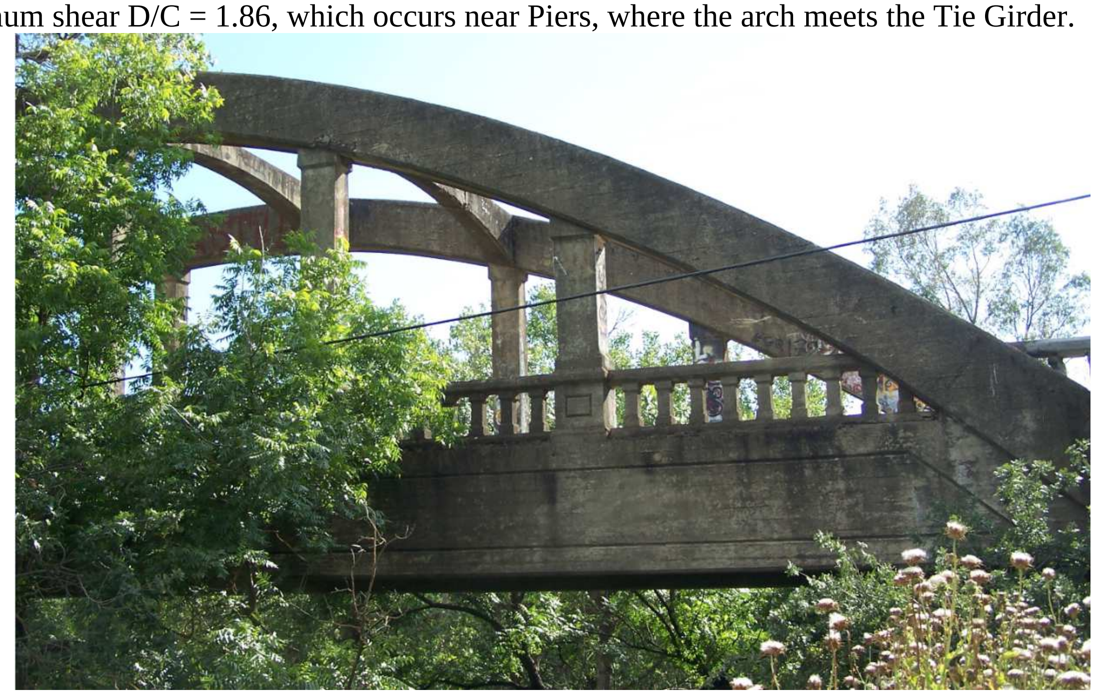
Tie Girders

The Tie Girder is a tension element that acts like a bow string that prevents the arch from flattening out. They also vertically support the floor beams with assistance from the vertical hangers. The Tie Girders are 63 inches deep by 23 inches wide. Per the As-built Plans, the tie girders are reinforced with twelve $1\frac{1}{8}$" square bottom bars and five $1\frac{1}{8}$" square top bars. The $1\frac{1}{8}$" square main bars are roughly equivalent to the area of a current #10 bar. For confinement, $\frac{1}{2}$" square bars at 18 inch spacing are provided. The $\frac{1}{2}$" square confinement bars are roughly equivalent to a current #4 bar. Like the arch rib, the existing amount of confinement is very minimal and does not meet today's minimum transverse reinforcement requirement. Without the minimum amount of transverse reinforcement, the Tie Girder is unable to provide much ductility in a seismic event.

Under earthquake loads, the Tie Girders are most vulnerable to transverse ground motions. Because these Tie Girders were primarily designed to handle vertical loads, they have very little strength in the transverse direction to handle lateral seismic forces. Lateral bending in the tie girder induced by frame action between the Vertical Hangers, Portal Bracing, and Floor Beams as well as direct transverse bending from the arch at the spring line near the piers result in a high transverse moment demand. The maximum bending for this behavior occurs near Pier 3 where the Tie Girder connects with the arch at the spring line.

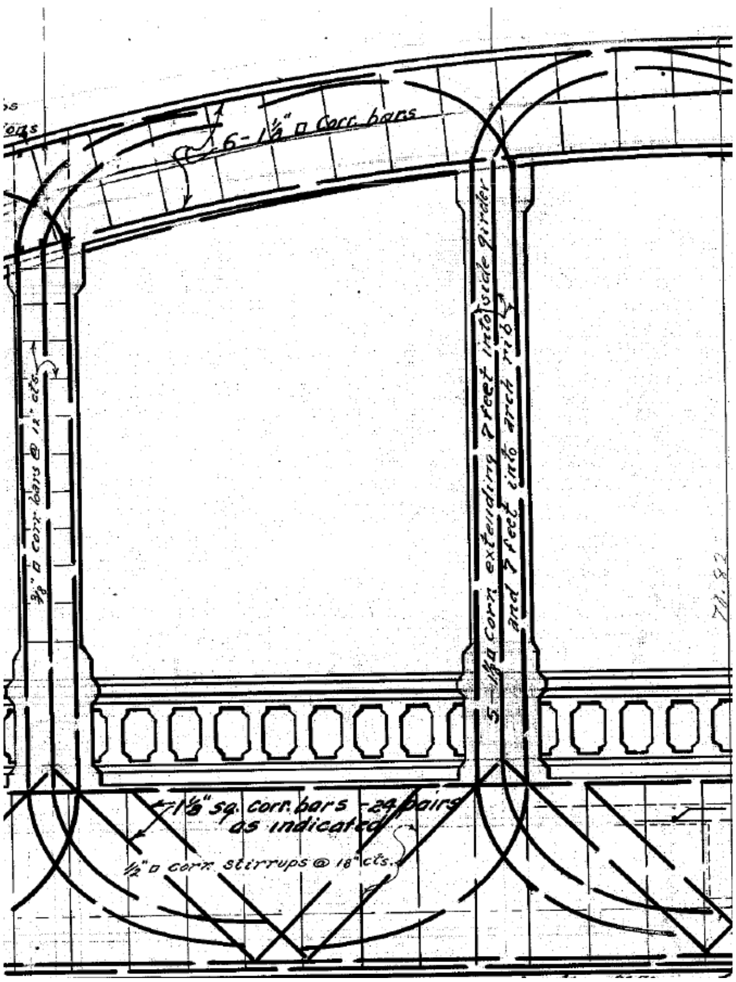
Analysis results indicate that the Tie Girders are also deficient in longitudinal flexure at the pier supports as they are not enough to resist plastic moments from the piers.

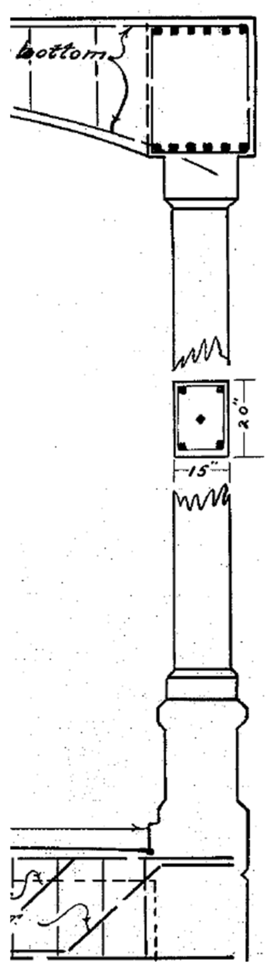
The maximum demand-to-capacity ratio of the Tie Girders for combined axial and flexural loads is 3.31.
The maximum shear D/C = 1.86, which occurs near Piers, where the arch meets the Tie Girder.



<u>Vertical Hangers</u>

The Vertical Hangers are 20 inches thick by 15 inches wide. Each hanger has five $1\frac{1}{8}$" square longitudinal reinforcing bars that extend seven feet into the Tie Girder, and seven feet into the Arch Rib. The $1\frac{1}{8}$" square bars have an area that is roughly equivalent to a current #10 bar. For confinement, $\frac{3}{8}$"square bars at 12 inch spacing are provided. The $\frac{3}{8}$" square bars are equivalent to today's #3 bars. The Vertical Hanger confinement does not meet the minimum transverse reinforcement requirements. Some of the existing Vertical Hangers already have significant cover loss in various locations, as shown in photos below. Based on the As-built plans, it is unclear if lap-splices were allowed during construction.





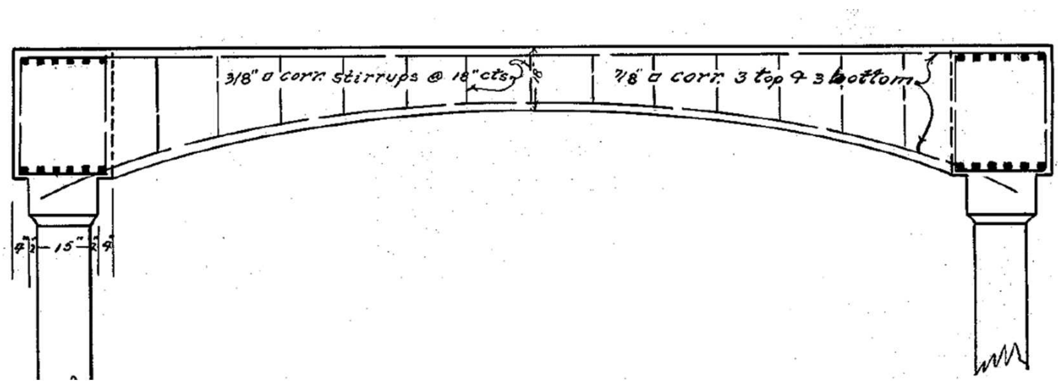
Under seismic loading, the Vertical Hangers remain in tension from carrying the bridge self-weight dead load. Because the hangers are in tension, their flexural capacity is very low.

The maximum demand-to-capacity ratio of the Vertical Hangers for combined axial and flexural loads is extremely overstressed, D/C >> 1.

The shear demand-to-capacity ratio is also extremely overstressed, D/C >> 1.

Portal Bracing

The Portal Bracing varies in depth between 36 inches deep at the end to 18” deep in the middle of the member. The braces are 20” wide. Each brace is reinforced with six $\frac{7}{8}$" square longitudinal bars on the top and bottom face. The $\frac{7}{8}$" square bars are equivalent to a current #7 bar. These bars are not adequately developed into the arches due to both the low concrete strength of the arch and the short distance the bars extend into the arch. This means the bars could pull out of the arch before they reach their maximum strength. Confinement reinforcement consists of 3/8” square bars at 18 inch spacing. The $\frac{3}{8}$" square bars are equivalent to a current #3 bar. Like most of the elements in the bridge, this rebar does not meet the minimum transverse reinforcement requirements.



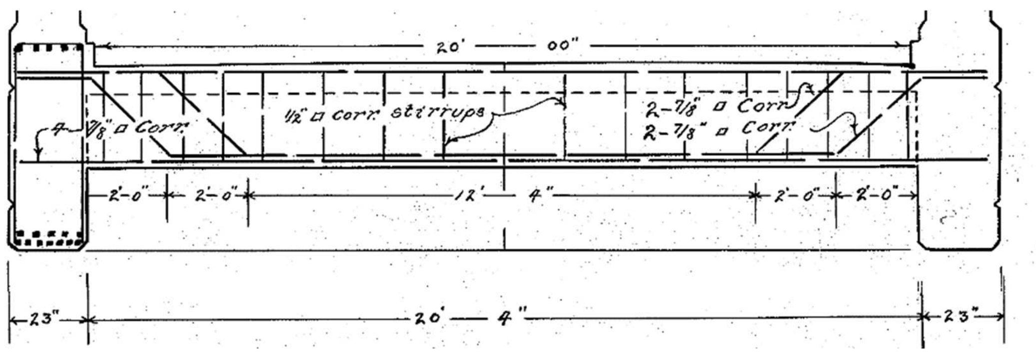
Portal Bracing is designed to resist transverse loading. When considered in conjunction with the adjacent Vertical Hanger, and Lower Transverse Floor Beam this frame makes up the primary transverse load resisting mechanism.

The maximum demand-to-capacity ratio of the Portal Bracing for combined axial and flexural loads is overstressed, D/C >> 1. The shear demand-to-capacity ratio, D/C = 0.95.

<u>Floor Beams</u>

The Floor Beams are 20'-4" long and 14" wide. The depth is not shown on the As-builts but is assumed to be 32" deep based on the relative As-built scale and site photos. They contain four $\frac{7}{8}$" square bars in the bottom mat near mid-span which are bent and extend to the top mat starting four feet away from the face of the Tie Girder. For confining shear steel $\frac{1}{2}$" square bars are spaced at 18" near mid-span and are spaced at a 12" spacing closer to the Tie Girder.

The maximum demand-to-capacity ratio of the Floor Beam for combined axial and flexural loads is overstressed, D/C = 1.06. The shear demand-to-capacity ratio D/C = 0.35. This overstress only occurs at floor beams located adjacent to the arch rib spring line.

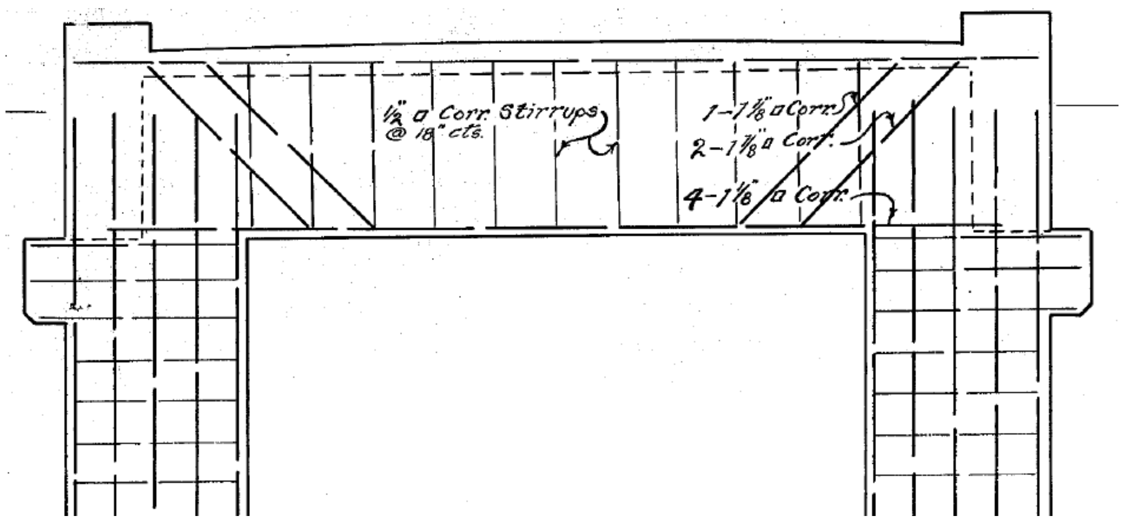


<u>Pier Cap</u>

The Pier Cap at Pier 3 is different than the existing Pier Cap system at Pier 2 and Pier 4.

At Pier 3, the existing Pier Cap size (not shown on the As-builts) has been approximated to be 4 feet deep by 2 feet wide. It appears to be deeper and thicker than the Floor Beams. The existing bent cap has six $1\frac{1}{8}$" square bars in the bottom mat near the mid span between the piers. As the bars approach the piers three bars are angled to the top mat at a 45 degree angle. The remaining four $1\frac{1}{8}$" square bars in the bottom mat extend over the pier supports. The As-built plans do not call out the number of bars in the top mat. Typical transverse deck bars have been assumed in the strength evaluation.

For confinement, $\frac{1}{2}$" square bars at 18 inch spacing are provided. The $\frac{1}{2}$" square confinement bars are equivalent to a current #4 bar.



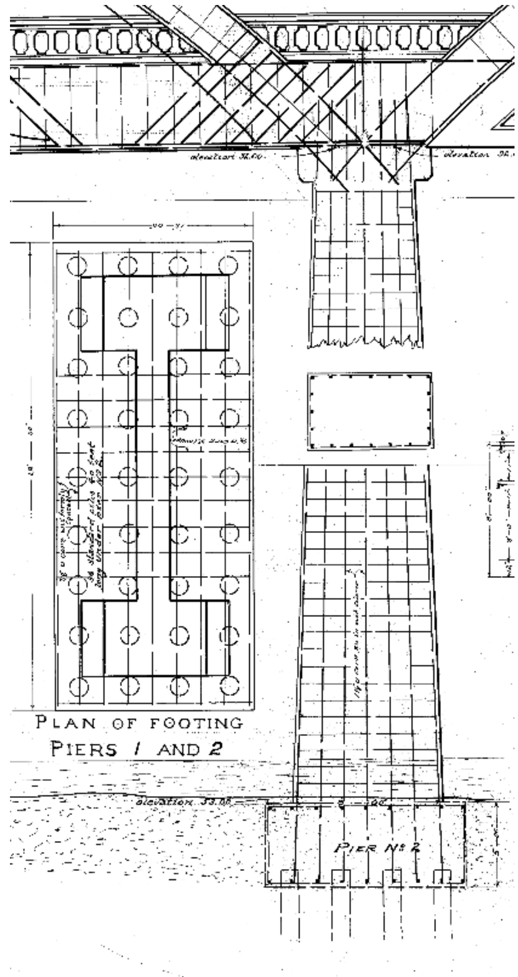
Pier 3 As-built Elevation

At Pier 2 and Pier 4 the existing Pier Cap is also approximately 4 feet deep by 2 feet wide, however they are different than Pier 3 because the Pier Cap is tied into the approach T-spans. While not explicitly shown on the As-built plans, Pier 2 and Pier 4 have been assumed to have the same rebar configuration as Pier 3 since the dimensions of the Pier Caps are the same.

The maximum demand-to-capacity ratio of the Pier Cap for combined axial and flexural loads is overstressed, D/C = 6.04. The shear demand-to-capacity ratio D/C = 2.38.

<u>Piers</u>

The existing pier columns are 4 1/2 feet thick, and their depth varies from 6 feet at the top deep by 9 feet deep at the bottom. The existing pier columns have $1\frac{1}{8}$" square bars as shown in the As-built plans. The $1\frac{1}{8}$" square confinement bars are equivalent to a current #10 bar. For confinement, $\frac{1}{2}$" square bars at 12 inch spacing are provided. The $\frac{1}{2}$" square confinement bars are equivalent to a current #4 bar.



The Piers behave inelastically in flexure in both the longitudinal and transverse directions, with demand-to-capacity ratio D/C >> 1.

The pier columns have a maximum seismic shear demand-to-capacity ratio of D/C = 0.75.

The pier column capacity and the seismic demands are reported in the table below for Pier 3. Pier 2 and Pier 4 had similar results.

Pier 3 Results						
	Force Demand [kips-inches] @ pier bottom					
	Design ARS Curve Response				Governing Demand Results	
	Case I, Trans + DL 100% Transverse + 30% Longitudinal		Case II, Long + DL 30% Transverse + 100% Longitudinal			
Longitudinal: Transverse:	36,700 k-in 126,200 k-in		94,200 k-in 38,300 k-in		94,200 k-in 126,200 k-in	
	Force Capacity [kip-inches] @ pier bottom					
	Axial Load, P	M-yield	Mp	Icrack [in^4]	M-yield	D/C
Longitudinal	700 k (comp)	72,400 k-in	92,200 k-in	1,398,000	72,400 k-in	1.30
Transverse	700 k (comp)	40,500 k-in	49,400 k-in	397,000	40,500 k-in	3.12

Pier 3 Existing Bridge Seismic Analysis Results

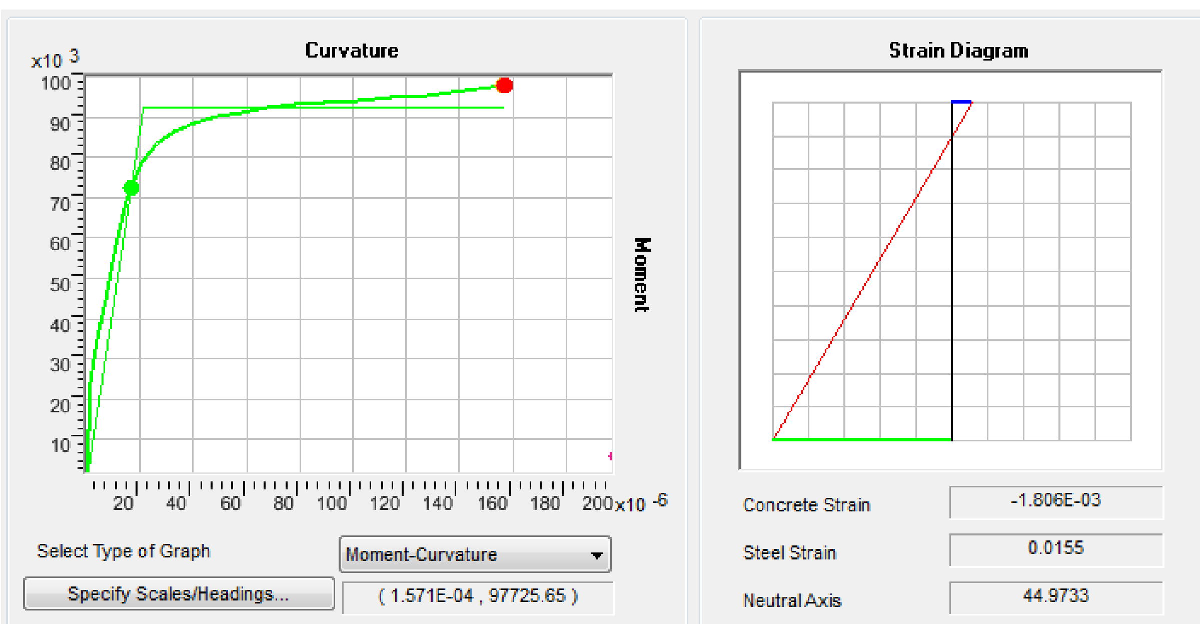
Since the pier behaves inelastically, Moment-Curvature $M-\phi$ analyses were performed with the expected material properties. The maximum moment capacity is determined when either the ultimate concrete compressive strain ε_{cu} or the reduced ultimate tensile strain ε_{su}^{R} of reinforcement steel is reached. The As-built plans do not identify if lap splices were used in the main reinforcement (typically at the top of the footing), a conservative concrete strain limit of 0.002 was used, and is the controlling factor in determining the displacement/curvature capacity for the nonlinear assessment. A reduced ultimate tensile strain ε_{su}^{R} of 0.06 was used for the reinforcement and is based on the uncertainty of the historical properties of reinforcement from the 1930's.

Comparing the Pier 3 displacement capacities to the displacement demands, the calculated longitudinal displacement capacity of the column is 3 inches, which is greater than the 1.0 inch of longitudinal displacement demand resulted from the response spectrum analysis. The Pier 3 longitudinal displacement demand-to-capacity ratio is D/C = 0.33.

For the transverse direction overturning effects were considered, the calculated transverse displacement capacity of Pier 3 is 3.8 inches, which is greater than the 3.2 inches of transverse displacement demand resulted from the response spectrum analysis. The Pier 3 transverse displacement demand-to-capacity ratio is D/C = 0.85.

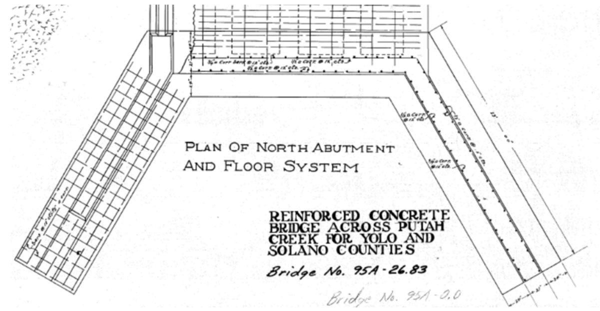
$P\Delta$ effects are negligible since the relative pier displacement multiplied by the dead load is small when compared to the column idealized plastic moment.

While the estimated displacement capacities are greater than the design displacement demand, fiber wrap of the Pier is still proposed due to the uncertainty in the existing reinforcing steel's ability to provide adequate confinement for the anticipated large strains.



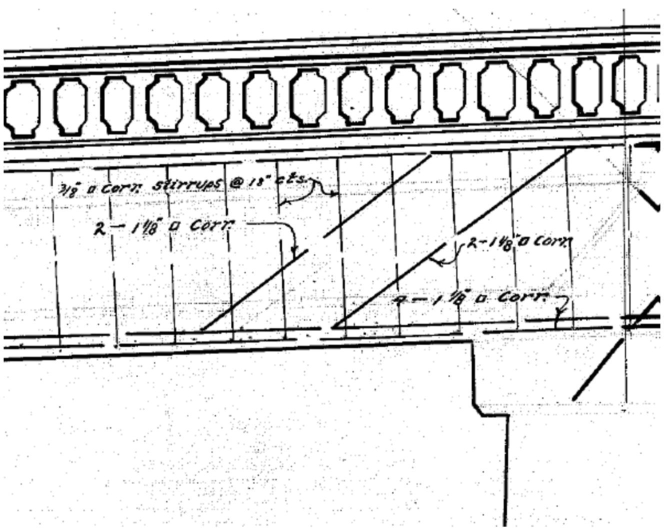
Pier 3 Moment-Curvature about the Weak Axis

Abutments



The South Abutment (Abutment 1) is approximately 12 feet tall and the North Abutment (Abutment 5) is approximately 6 feet taller. Both abutments consist of diaphragms 21 inch wide and are supported by spread footings 24 inches deep. Although not shown on the As-builts, there is a vertical construction joint between the wingwall and diaphragm. Also not shown are large concrete leveling pads cast below the abutment footing. The large concrete leveling pads were visible during our site visits. While analysis has shown that the lightly reinforced diaphragms will fail in bending and shear just below the soffit for seismic loading, this failure would not likely result in a collapse mechanism. Consequently, no retrofit measures are recommended for this deficiency, however retrofit piles are still proposed at the abutments to support the abutments for scour, future settlement, and add stiffness in the longitudinal direction to reduce seismic demands.

Approach Spans



The approach spans (spans 1 & 4) are 40 feet long and consist of five concrete "T" beams. Large transverse deck cracks have been observed in both approach spans. Cracks for both spans occur about 3/4 of the way along the span (closer to the pier) which also correlates closely to the where the As-built plans show that the Tie Girder negative moment steel angles down towards the bottom fiber. The location of the cracks would suggest that both abutments have settled over the life of the bridge, or that the structure experiences a larger negative moment in the approach span near the support than originally considered in design. Other than these cracks which appear stable and not progressing no other deficiencies have been identified.

Summary of Deficiencies

In summary, the bridge has numerous deficiencies as discussed above. Most of these members must be retrofitted, and the retrofit strategy is discussed in the following pages.

8. RETROFIT ALTERNATIVES

The previous seismic strategy prepared by TRC considered two primary retrofit alternatives. The major difference between the alternatives were inclusion and exclusion of the proposed cast-in-drilled hole (CIDH) concrete piles behind the abutments. Given the scour elevations at the abutments provided by WRECO in conjunction with the observed existing settlement at both abutments, a retrofit alternative that does not include CIDH piles at the abutment is no longer feasible.

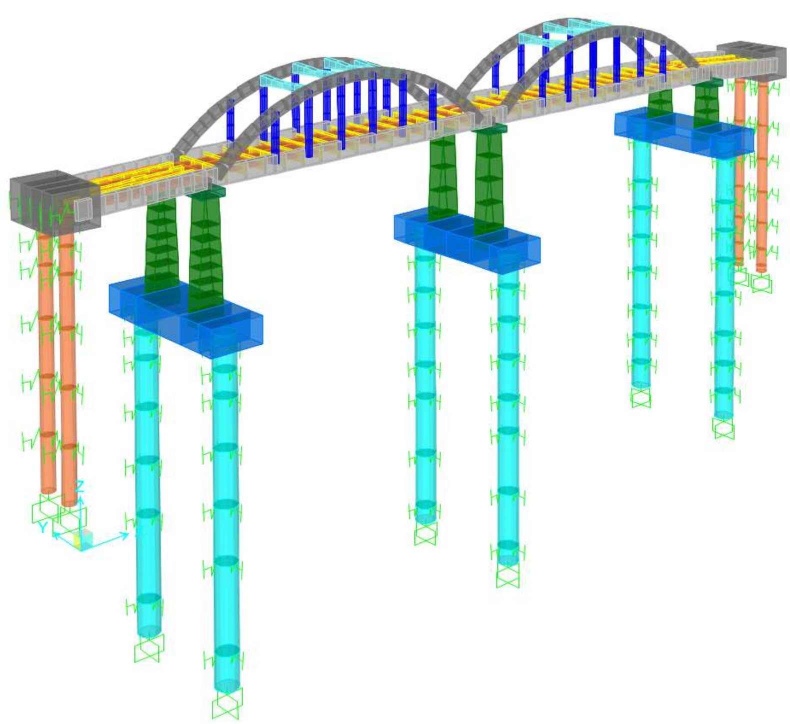
While two distinct retrofit alternatives are not presented in this report, Quincy evaluated numerous alternatives and combinations of alternatives in order to present the highest performance retrofit at the lowest cost. Quincy developed over twelve retrofit models with various boundary fixity conditions. Quincy also looked at the structural effects of including and excluding infill walls between each pier in order to fully understand what stiffness and boundary conditions result in the least amount of superstructure retrofit.

Since all piers and abutments require retrofit due to scour concerns regardless of seismic performance, the most cost effective retrofit strategy is one that minimizes superstructure demands. In general, increasing the stiffness in the longitudinal direction, and having a lower stiffness in the transverse direction resulted in the lowest superstructure demands based on our sensitivity analysis. This retrofit strategy is presented below and is recommended for final PS&E.

9. RETROFIT STRATEGY

Numerous retrofit measures must be incorporated in order to address the deficiencies summarized above. Please note that this retrofit strategy addresses seismic and scour deficiencies for a no collapse criteria only. Live load analysis for service and strength loads were not evaluated.

The concrete arch bridge is an unusual, complex structure that does not lend itself to common retrofit measures such as strengthening members by encasing them in concrete or steel jackets, or constructing in-fill walls between members. In order to maintain the general appearance of the bridge for historical considerations, strengthening deficient members by fiber wrapping is proposed as one of the primary retrofit measures for visible members. Fiber wrapping material provides additional strength and confinement/ductility with a minimal change to the dimensions of the member. More conventional strengthening using concrete and steel is also proposed at less visible locations such as the interior side of the Tie Girder and at the Pier Caps.



Substructure retrofit measures include the retrofit of all abutment and pier footings to resist both scour and seismic deficiencies. Large diameter piles would be added to the outside of the existing pier footings and a new pier footing cap would tie the new piles to the existing footing. At the abutments, large diameter piles would be added behind the existing abutment wall to address both scour and seismic deficiencies.

The proposed retrofit strategy strengthens primary superstructure members (Arch Rib & Tie Girders) to remain elastic during a seismic event. Secondary members (portal, vertical hanger and floor beams), as well as the piers may experience some inelastic behavior, however fiber wrap is proposed to increase ductility and prevent collapse. The arch rib, tie girders and pier caps will be strengthened in order to resist plastic moments from the piers which insures the primary members in the superstructure will remain elastic. The retrofit of each element is discussed in further detail below.

Fiber Reinforced Polymer Retrofit

Fiber Reinforced Polymer (FRP) provides additional strength and ductility to bridge elements. Caltrans has approved FRP for use in jacketing various structural members to increase their strength, and FYFE Company LLC is one of the companies that have been preapproved by Caltrans to do such work. Below is a FYFE product specification for the SCH-41 Carbon system (CT system 9) approved by Caltrans. The retrofit strategy mentioned in the following pages utilizes this carbon fiber wrap system to strengthen various arch elements.

FRP is commonly been used to provide confinement, and axial and shear capacity enhancement for existing members. FRP can also provide additional flexural capacity to members.

The design guidelines for FRP strengthening are presented in ACI 440.2R-08 *"Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures"*. Additional information on the criteria for evaluation fiber wrap systems can be found in International Code Council's ICC-ES AC 125 *"AC125 Concrete and Reinforced and Unreinforced Masonry Strengthening Using Fiber-reinforced Polymer (FRP) Composite Systems"*.

FYFE
Tyfo® FIBRWRAP® Systems

ISO 9001:2008 REGISTERED FIRM

Tyfo® SCH-41 Composite
using Tyfo® S Epoxy

DESCRIPTION
The Tyfo® SCH-41 Composite is comprised of Tyfo® S Epoxy and Tyfo® SCH-41 reinforcing fabric, which is NSF-Certified. Tyfo® SCH-41 is a custom, uni-directional carbon fabric orientated in the 0° direction. The Tyfo® S Epoxy is a two-component epoxy matrix.

USE
Tyfo® SCH-41 Fabric is combined with Tyfo® Epoxy to add strength to bridges, buildings, and other structures.

ADVANTAGES
- ICC-ES ESR-2103 listed product
- Component of UL listed, fire-rated assembly
- NSF/ANSI Standard 61 listed product for drinking water systems
- Improved long-term durability
- Good high & low temperature properties
- Long working time
- High tensile modulus and strength
- Ambient cure
- 100% solvent-free
- Rolls can be cut to desired widths prior to shipping

COVERAGE
Approximately 600 sq. ft. surface area with 3 to 4 units of Tyfo® S Epoxy and 1 roll of Tyfo® SCH-41 Fabric when used with the Tyfo® Saturator.

PACKAGING
Order Tyfo® S Epoxy in 55-gallon (208L) drums or pre-measured units in 5-gallon (19L) containers. Tyfo® SCH-41 Fabric typically shipped in 24" x 300 lineal foot (0.6m x 91.4m)

TYPICAL DRY FIBER PROPERTIES

PROPERTY	TYPICAL TEST VALUE
Tensile Strength	550,000 psi (3.79 GPa)
Tensile Modulus	33.4 x 10^6 psi (230 GPa)
Ultimate Elongation	1.7%
Density	0.063 lbs./in.^{3} (1.74 g/cm^3)
Minimum weight per sq. yd.	19 oz. (644 g/m^2)

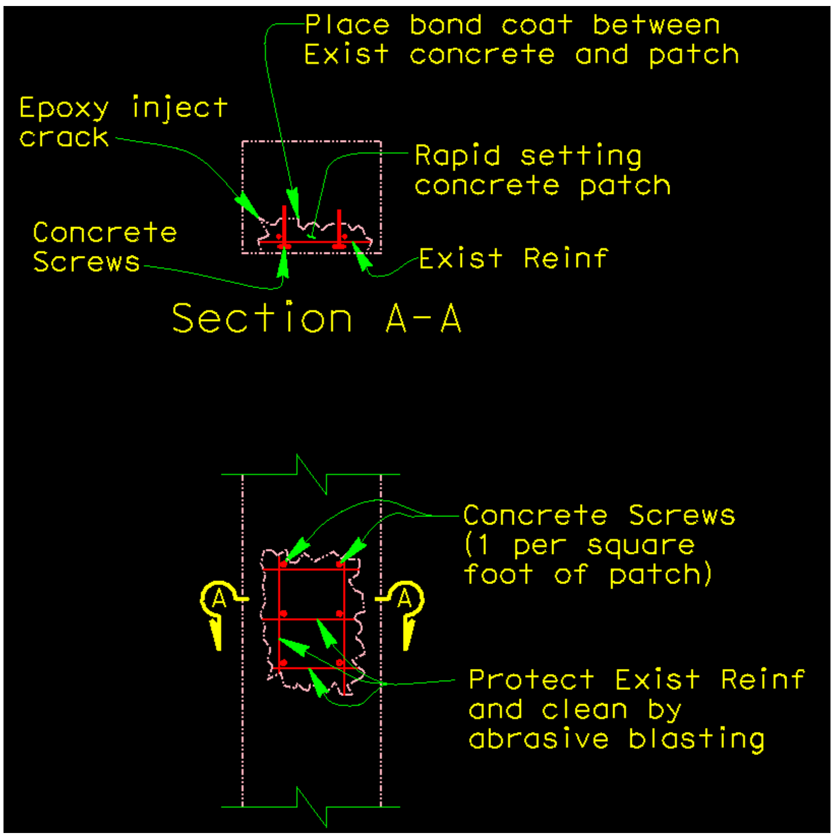
COMPOSITE GROSS LAMINATE PROPERTIES

PROPERTY	ASTM METHOD	TYPICAL TEST VALUE	DESIGN VALUE*
Ultimate Tensile Strength in Primary Fiber Direction	D3039	143,000 psi (986 MPa) (5.7 kip/in. width)	121,000 psi (834 MPa) (4.8 kip/in. width)
Elongation at Break	D3039	1.0%	0.85%
Tensile Modulus	D3039	13.9 x 10^6 psi (95.8 GPa)	11.9 x 10^6 psi (82 GPa)
Flexural Strength	D790	17,900 psi (123.4 MPa)	15,200 psi (104.8 MPa)
Flexural Modulus	D790	452,000 psi (3.12 GPa)	384,200 psi (2.65 GPa)
Longitudinal Compressive Strength	D3410	50,000 psi (344.8 MPa)	42,500 psi (293 MPa)
Longitudinal Compressive Modulus	D3410	11.2 x 10^6 psi (77.2 GPa)	9.5 x 10^6 psi (65.5 GPa)
Longitudinal Coefficient of Thermal Expansion	D696	3.6 ppm./°F	
Transverse Coefficient of Thermal Expansion	D696	20.3 ppm./°F	
Nominal Laminate Thickness		0.04 in. (1.0mm)	0.04 in. (1.0mm)

Caltrans pre-approved Carbon Fiber Wrap
(Tyfo SCH-41 Composite system)

<u>Surface Preparation</u>

Before installing FRP, the surface of the member must be prepared. First, unsound concrete must be removed and replaced. Corrosion on bars should be removed by abrasive blasting (water blasting may also be feasible). Then the surface is repaired by injecting epoxy into any cracks in the concrete. For surfaces requiring repair with an area greater than one square foot, concrete screws should be installed to provide better bond to the new concrete. Next, all concrete surfaces to be wrapped with FRP should be abrasive blast cleaned or ground to provide a rough bonding surface. Corners of the FRP retrofitted members must also be rounded to a minimum radius of 2” so that a sharp corner does not induce high stresses in the FRP, that could cause it to fail.



RFP Maintenance/Appearance

The FRP carbon fiber system itself is susceptible to decay due to ultraviolet exposure. To mitigate this effect and prolong the retrofit system, the FRP must be painted. The paint system is also susceptible to ultraviolet exposure and weathering, so it is necessary to repaint the FRP every 10 years to maintain the protective coating. Without intermittent maintenance, the FRP will eventually lose structural capacity. Because of the need to provide ongoing maintenance of the protective coating, the County's future cost to maintain the retrofitted bridge is higher compared to maintaining a new concrete bridge. In addition, the FRP is susceptible to damage from vehicles hitting/scraping the areas exposed to traffic on the narrow bridge. This is especially a concern with the wide agricultural equipment moving throughout the County.

The FRP and the paint will affect the appearance of the retrofitted bridge. Technically, only the portions of the bridge that have FRP installed require painting, which will result in an inconsistent appearance of the bridge. This could be addressed by painting the entire bridge.

Applying the FRP system to elements such as the vertical hangers will require the alteration of architectural column cap and base details, as well as the guardrail, to fully wrap the structural element. Additionally, it should be noted that the corners of any FRP wrapped elements will have to be rounded (to approximately two inch radius) to apply the fiber wrap, which will also alter the appearance of the bridge. Finally, the FRP will also cover portions of the architectural detailing (grooves) in the exterior face of the Tie Girder. Once the PS&E is finalized, details that modify the structure appearance should be reviewed to ensure they are consistent with the visual impacts discussed in the environmental documents for this historic structure.

Potential Retrofit Risks

The appearance of some architectural features of the bridge will potentially be adversely affected. The extent of the visual impacts are generally understood, but will not be fully known until the actual details are finalized in the developed design phase of the project.

With the relative newness of the proposed retrofit technology for this type of structure, it is possible that project costs could increase significantly as the details are developed during the final design phase of the project.

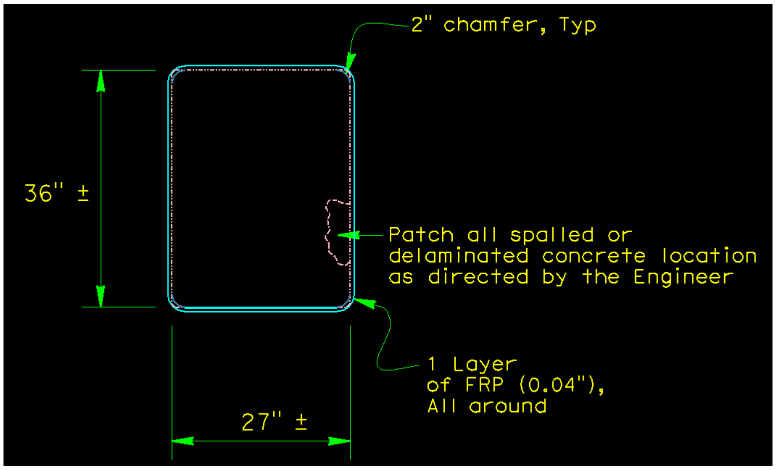
While we believe fiber wrap is the best alternative to retrofit the existing structure without changing the appearance, long-term durability of fiber wrapped structures is not well defined. Fiber wrap has only been used on bridges over the past 25 years, therefore there is an element of risk in estimating the design life of a bridge retrofitted with this technology.

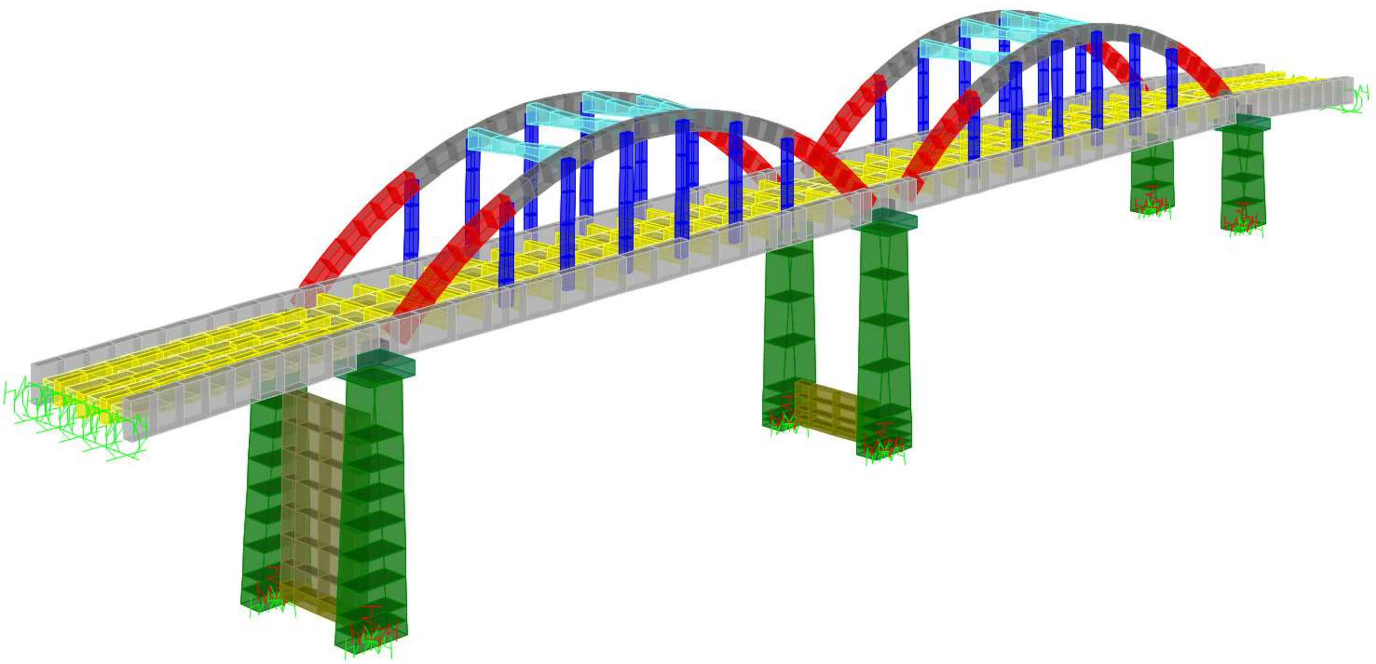
Furthermore, as stated previously, long term performance of the retrofitted bridge could deteriorate without proper County maintenance. The fiber wrapped portions of the bridge will need to be repainted periodically at a future cost to the County. In addition, any repairs to the fiber wrap caused by damage from vehicular impacts would also be an added expense to the County.

<u>Arch Rib Retrofit</u>

Due to the Arch Ribs failing in flexure, they will require retrofitting at the spring line (see areas shown in red below. The retrofit will be comprised of:

- At the outside face, the FRP will start at the spring line and end at the first vertical hanger.
- The arch will be wrapped with one layer of FRP (0.04") to provide additional confinement.
- Remove unsound concrete and patch spalls

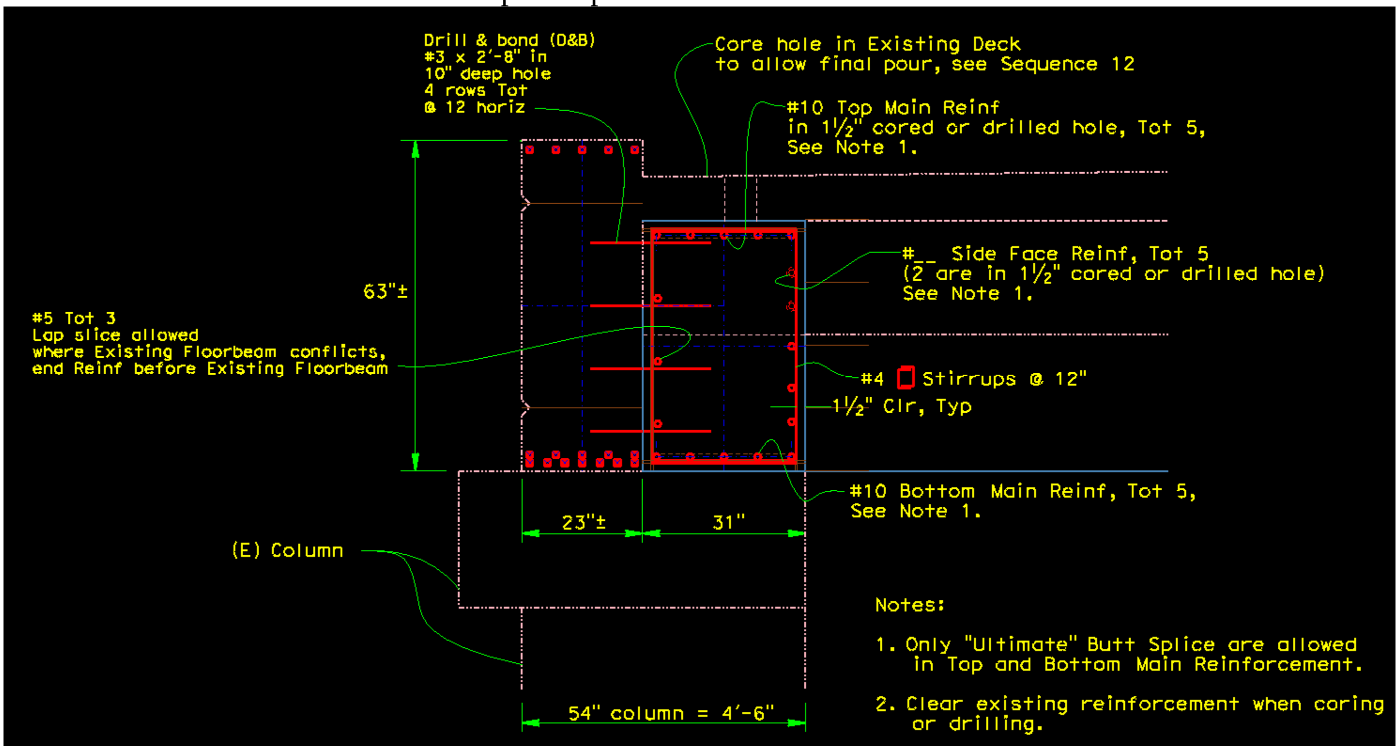




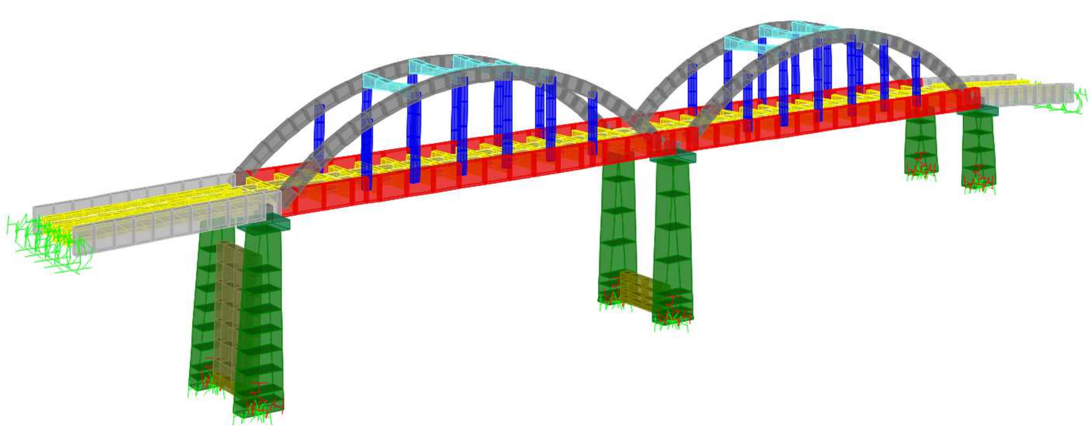
Tie Girder Retrofit

The Tie Girders are deficient in flexure at the pier supports and portions of the spans. The pier retrofit strategy will allow for inelastic behavior, therfore it is prudent to verify that the Tie Girder and Arch Rib connection can accommodate platic moments coming from the pier. One benefit to this locaition is that a coventional bolster using concrete and steel would be hidden from public view and therefore not adversely affect the historic resource. The proposed retrofit is to widen the interior side of the Tie Girder along the entire length in order to add additional flexure capacity. The retrofit will be comprised of:

- Enlarge the Tie Girder with a concrete bolster to the inside of the girder
- Remove unsound concrete and patch spalls



Red elements in the figure below indicate the approximate location of where the Tie Girder will be retrofitted with section enlargement. The retrofit model did account for increased mass as a result of this retrofit.

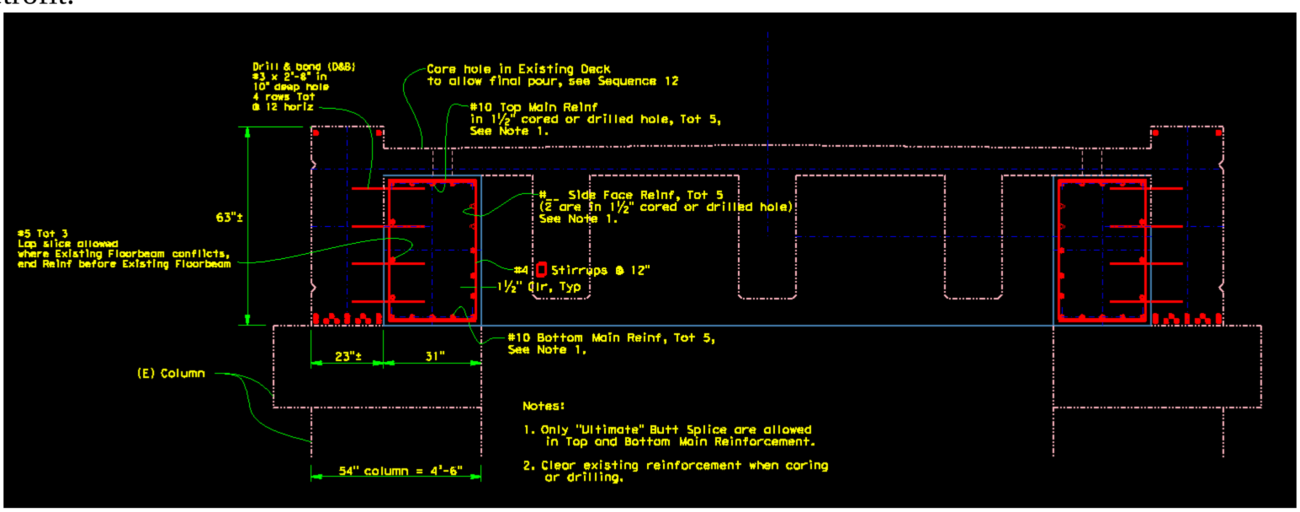


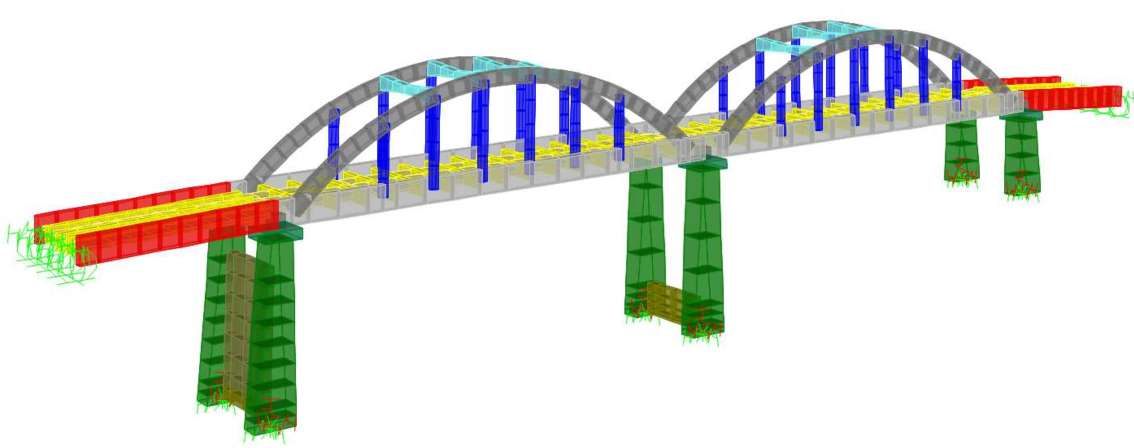
<u>Approach Span Exterior Girder Retrofit</u>

The Approach Span Exterior Girders (which is an extension of the Tie Girder from the Arch spans) must be strengthened to resist the overstrength moment demand of the pier sections and bending at the abutment diaphragm. The pier retrofit strategy will allow for inelastic behavior, therfore it is prudent to verify that the Approach Span Exterior Girders and Arch Rib connection can accommodate platic moments coming from the pier. Similar to the Tie Girder, this locaition is bennefited from the fact that a coventional bolster using concrete and steel will be hidden from public view and therefore will not adversely affect the historic resource. The proposed retrofit is to widen the interior of the Approach Span Exterior Girders along the entire length from the abutment to the pier location, which will add additional capacity. This retorift in conjunction with the Tie Girder retrofit, also creates a stronger continuous strut along the entire bridge from abutment to abutment. This helps transfer seismic loads from the structure into the abutment soil and retrofit piles behind the abutment. The retrofit will be comprised of:

- Enlarge the Approach Span Exterior Girders with a concrete bolster to the inside of the girder at each support (continous from the abutments to the pier)
- Epoxy inject transverse deck cracks
- Remove unsound concrete and patch spalls

Red elements in the figure below indicate the location of where the Approach Span Exterior Girders will be retrofitted with section enlargement. The retrofit model did account for increased mass as a result of this retrofit.



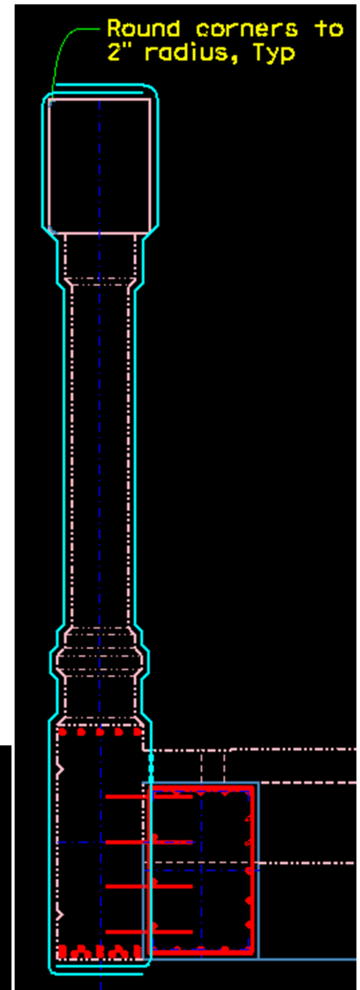


Vertical Hanger Retrofit

Due to the Vertical Hangers failing in both flexure and shear, all verticals will require retrofitting. Two alternative retrofits have been presented below. Unlike previous retrofits described above, the Vertical Hangers may not remain elastic during seismic loads, however increased ductility will be provided with the retrofit in order to prevent collapse.

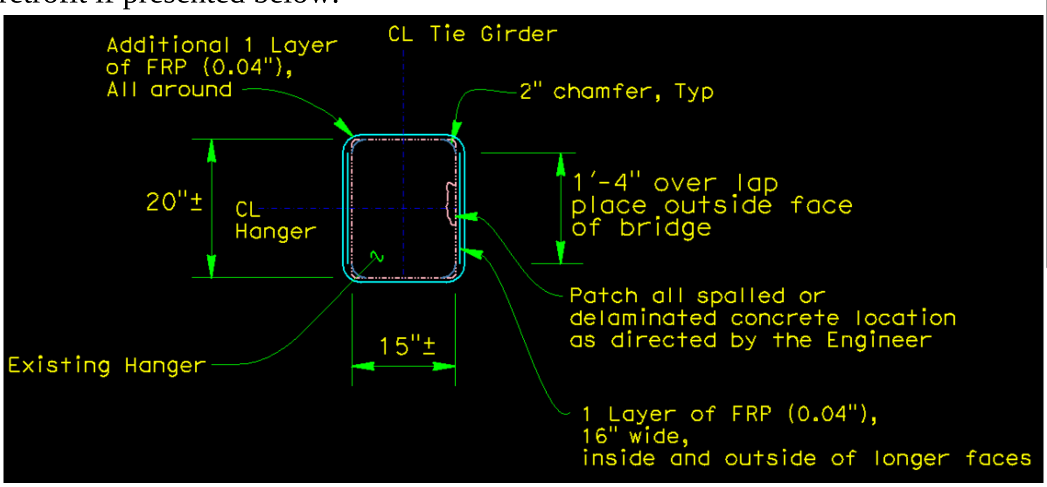
Vertical Hanger Retrofit Alternative 1 - FRP

This retrofit includes one layer of 0.04” FRP placed on the exterior and interior vertical face of the hanger in the plane of the arch. The FRP will be applied full height of the Vertical Hanger for strength. On the top it will extend over the top of the Arch, and on the bottom it will extend over the bottom of the Tie Girder. On the interior side, holes will be drilled through the deck to feed the FRP material through the deck so the FRP can be wrapped around the bottom of the Tie Girder. At locations were the Hanger has architectural features, a 4:1 slope of epoxy/mortar will be constructed to create a smooth transition for the FRP to reduce stress concentration.



Elevation of Vertical FRP Retrofit

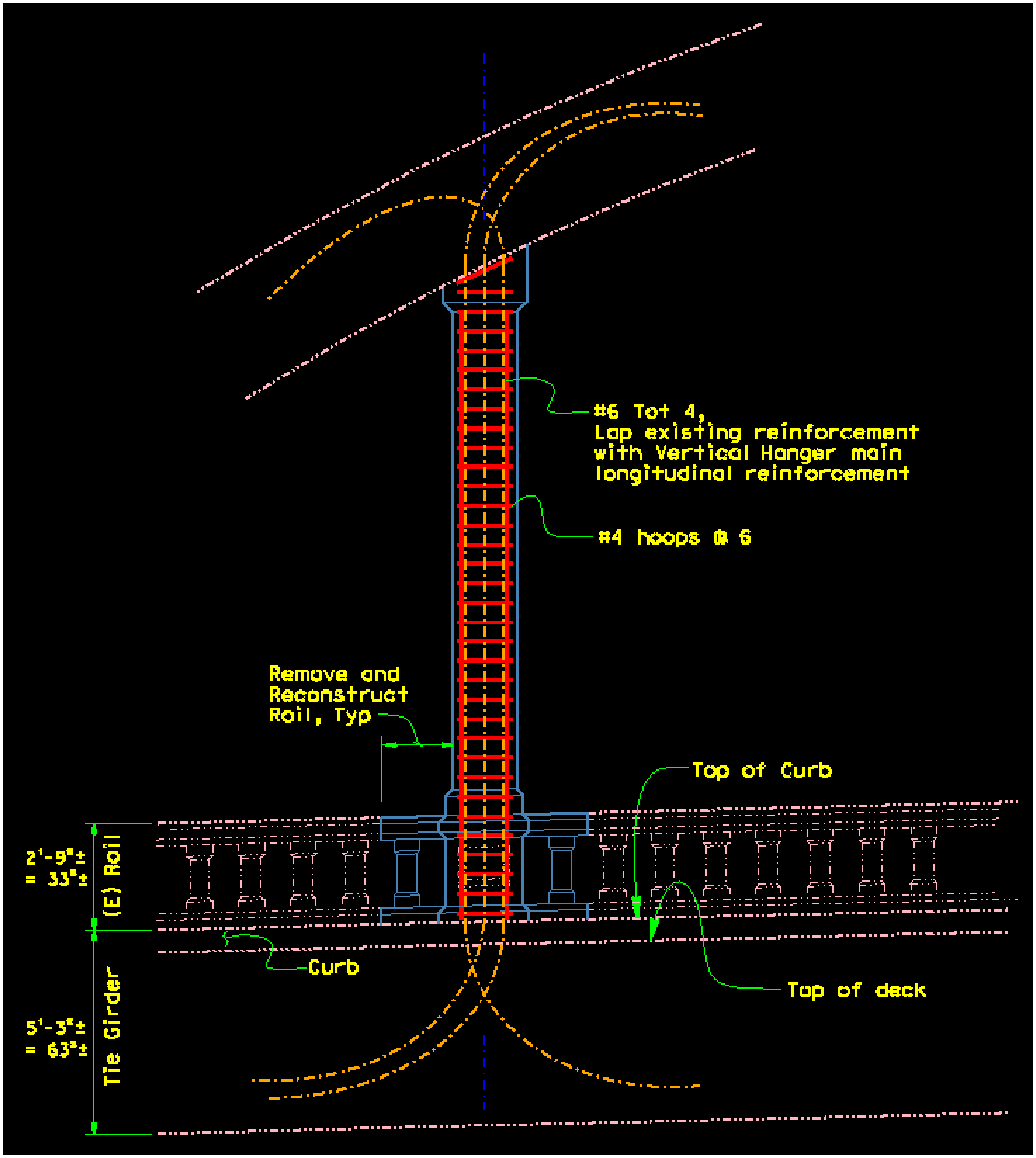
After the exterior and interior face FRP is applied to the Vertical Hanger, it will be wrapped horizontally around the perimeter for confinement. Corners of the Vertical Hanger will be chamfered round to eliminate stress concentration. To provide full element length confinement, the bridge railings will be removed adjacent to the verticals (except for the horizontal reinforcing bars) so that the FRP can be wrapped to the bottom of the Vertical Hanger. After the hanger is wrapped, the bridge railing will be reconstructed. This alterative does present some risk as the architectural features of the historic bridge must be modified for the FRP to be effective. Therefore, a second contingency vertical hanger retrofit if presented below.



Section of Vertical Hanger FRP Retrofit

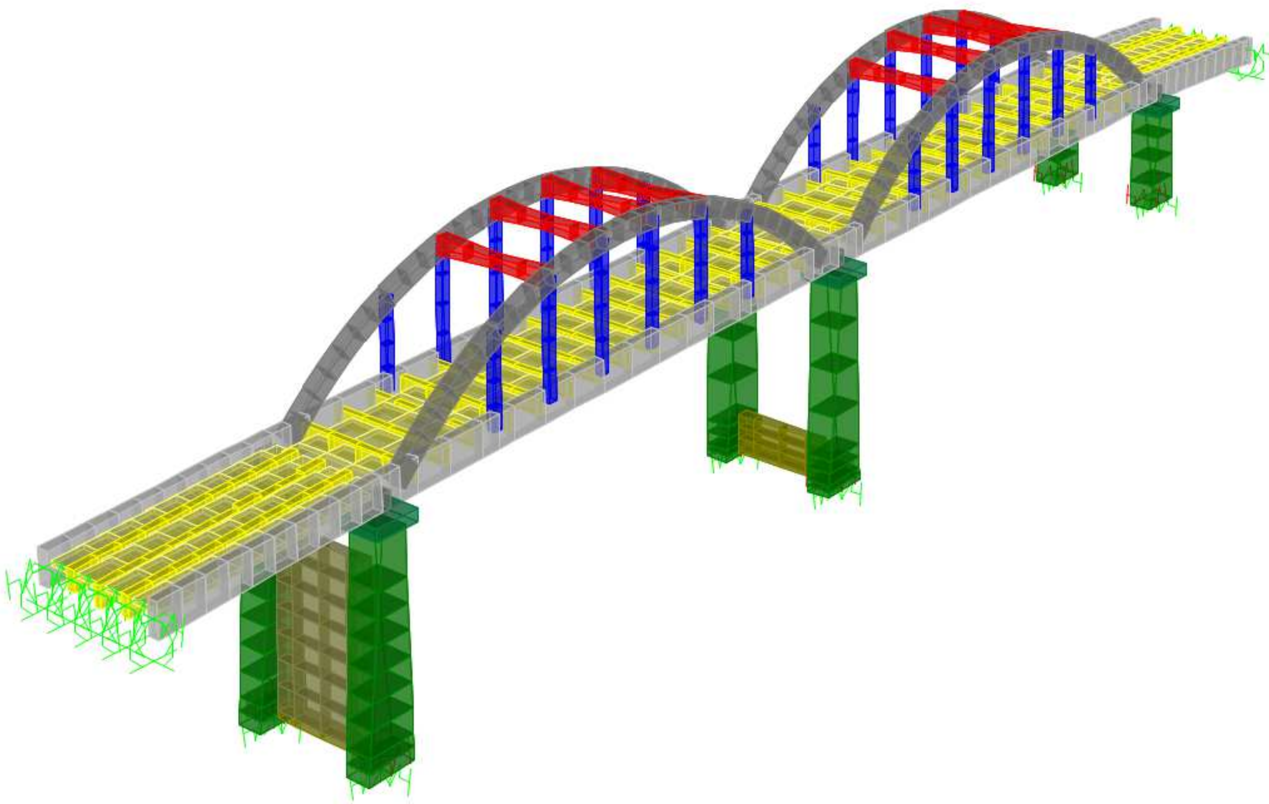
Vertical Hanger Retrofit Alternative 2 - Replacement

Replacement of the vertical hanger will be more difficult and expensive than retrofitting them with FRP, however the architectural shape can be matched exactly for historic considerations. To replace the Vertical Hangers, portions of the adjacent rail must be removed (similar to the FRP retrofit) to construct new concrete forms for the Vertical Hanger. In addition, the superstructure would have to be temporary supported with a falsework system from the ground or the arch. The falsework system must provide adequate vertical load caring capacity to hold up the bridge without the Vertical Hanger. The Vertical Hanger concrete would be removed while the existing bar reinforcing steel (rebar) is remains in place. The existing rebar would then be lapped with four new #6 rebars and #4 spiral ties at 6" pitch.

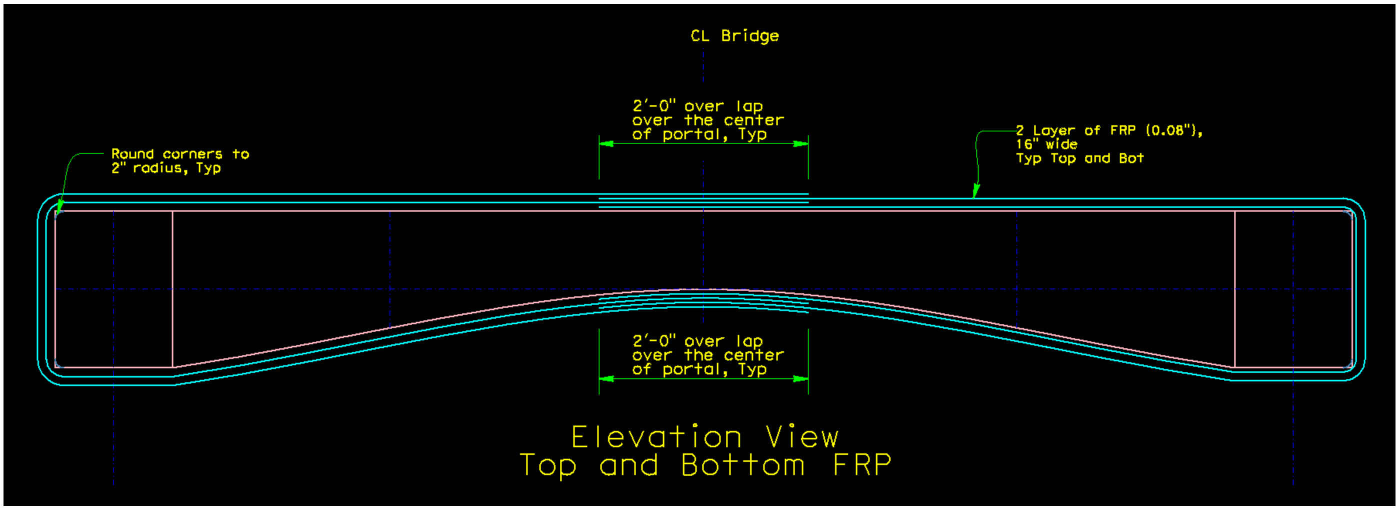


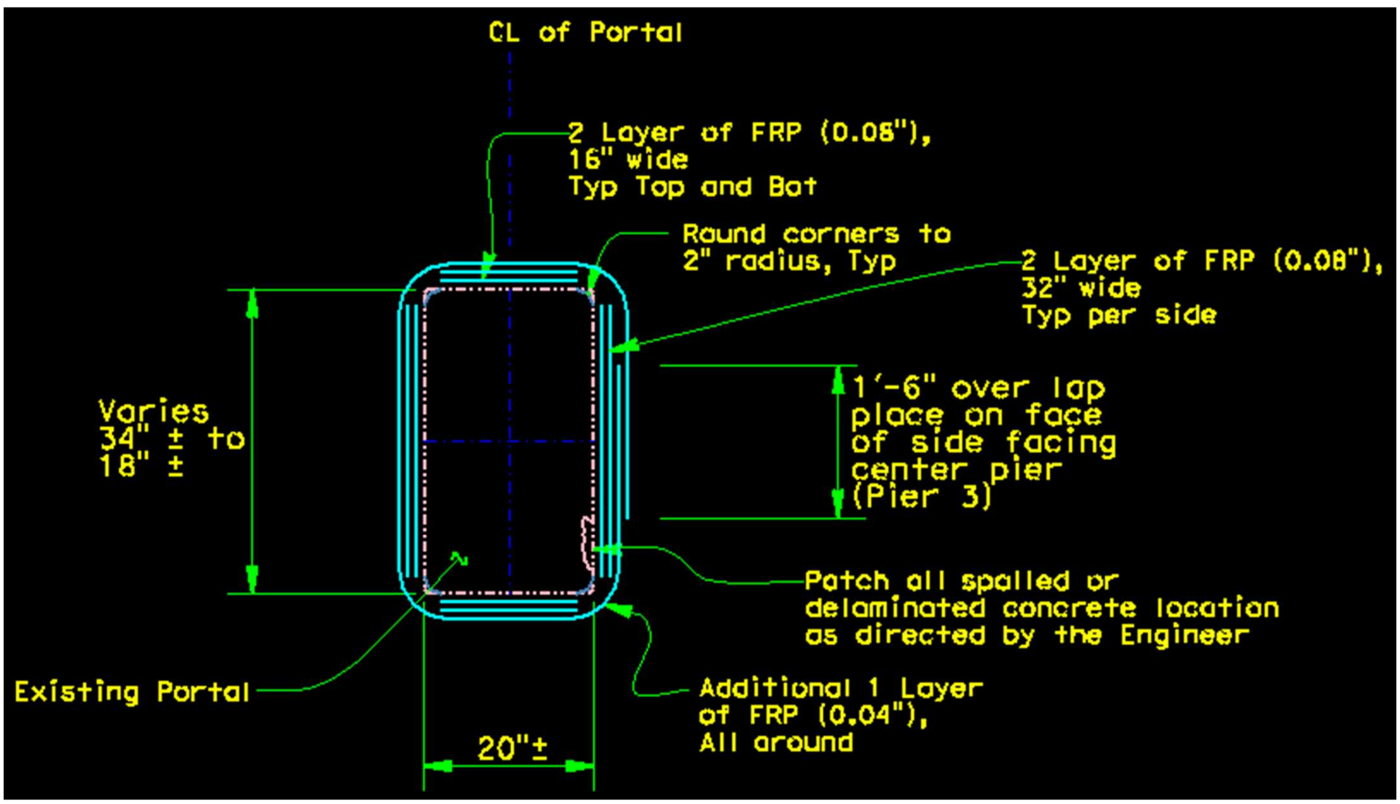
Portal Bracing Retrofit

Since analysis shows that the portal bracing is overstressed in flexure and is nearly overstressed in shear, the Portal Braces must be retrofitted. Similar to the Vertical Hangers, the Portal Braces may not remain elastic under seismic loading, however increased ductility will be provided in order to prevent collapse. The Portal Brace retrofit entails applying 2 layers of 0.04” FRP to each face along the length of the member. For the top and bottom layers, the FRP will be applied over the Arch Ribs as shown in the elevation view on the following page. In addition, the brace will be wrapped with 1 layer of FRP to provide additional confinement.



Limits of Portal Bracing Retrofit shown in Red

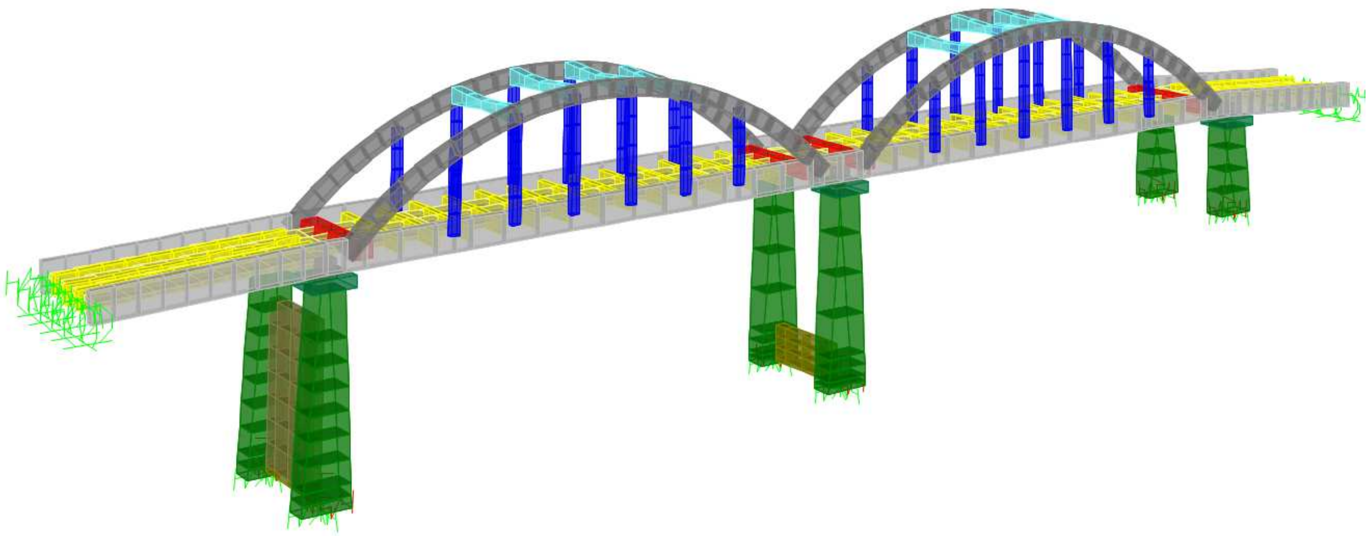

CL Bridge
2'-0" over lap
over the center
of portal, Typ
Round corners to
2" radius, Typ
2 Layer of FRP (0.08"),
16" wide
Typ Top and Bot
2'-0" over lap
over the center
of portal, Typ
Elevation View
Top and Bottom FRP


CL of Portal
2 Layer of FRP (0.08"),
16" wide
Typ Top and Bot
Round corners to
2" radius, Typ
2 Layer of FRP (0.08"),
32" wide
Typ per side
Varies
34" ± to
18" ±
1'-6" over lap
place on face
of side facing
center pier
(Pier 3)
Patch all spalled or
delaminated concrete location
as directed by the Engineer
Existing Portal
Additional 1 Layer
of FRP (0.04"),
All around
20"±

Floor Beam Retrofit

The Floor Beams adjacent to the Arch Ribs are deficient in flexure. They provide framing action and transverse rigidity and must be retrofitted for the floor beams to remain elastic and resist pier plastic loads.

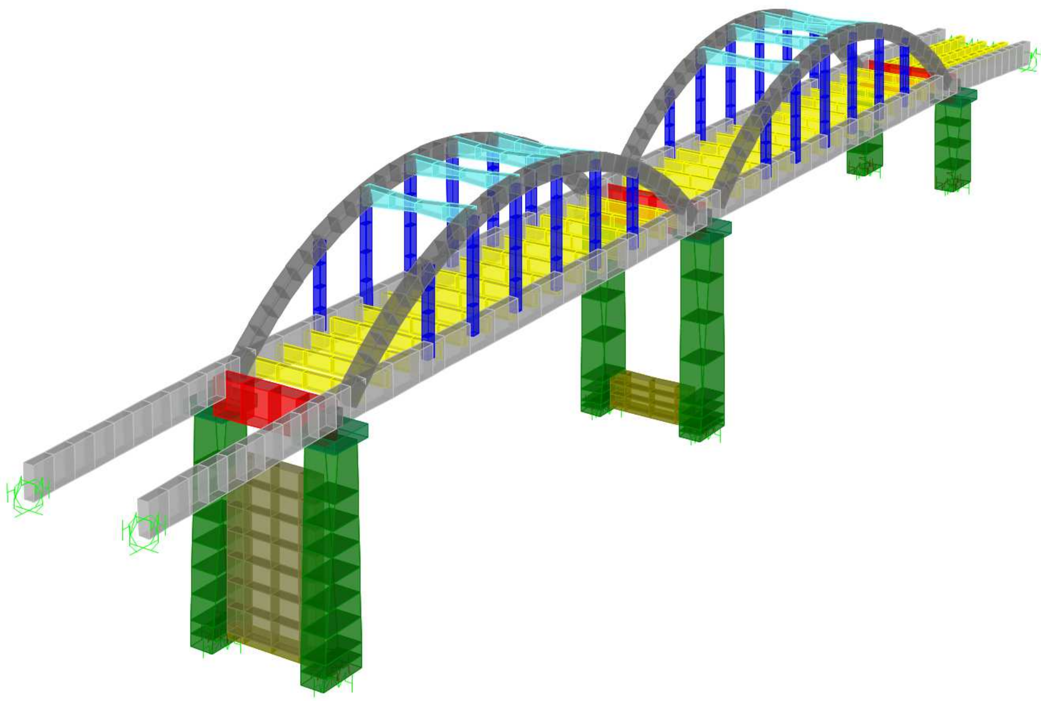
- Enlarge the Floor Beams are with a concrete bolster on both sides of the Floor Beams near each Arch Rib.



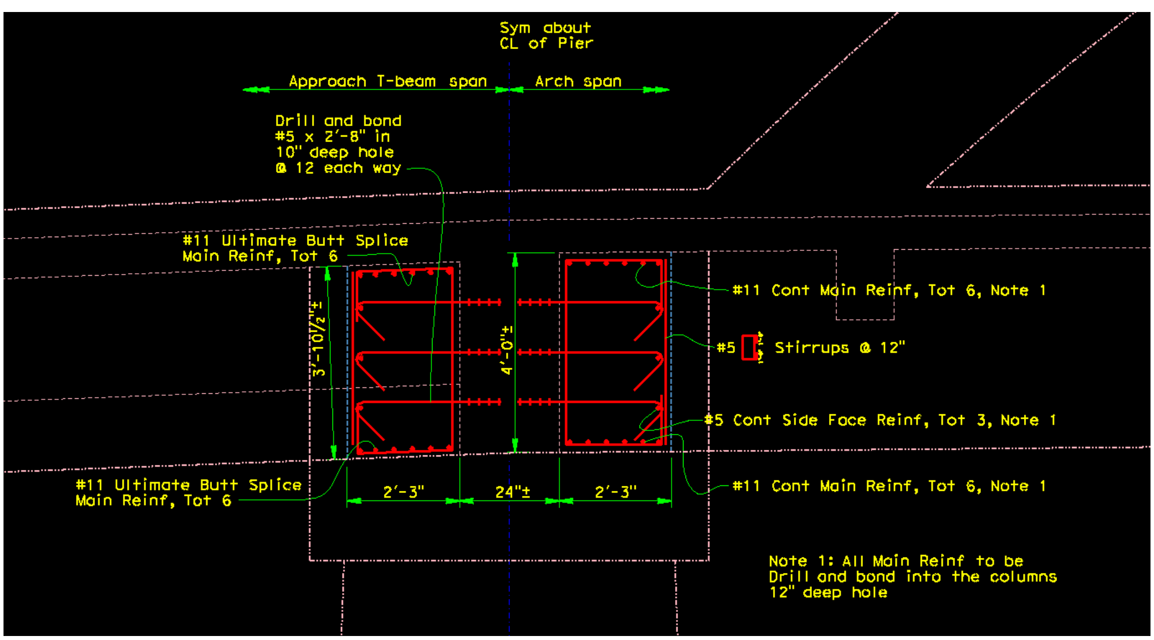
Limits of Floor Beam Retrofit shown in Red

Pier Cap Retrofit

The Pier Caps are deficient in flexure as the bridge moves transverely. The pier retrofit strategy will allow for inelastic behavior, therfore it is prudent to verify that the Pier Cap and its connection can accommodate platic moments coming from the pier. One benefit to this locaition is that a coventional bolster using concrete and steel would be hidden from public view and therefore not adversely affect the historic resource. The proposed retrofit is to thicken the pier cap beam.



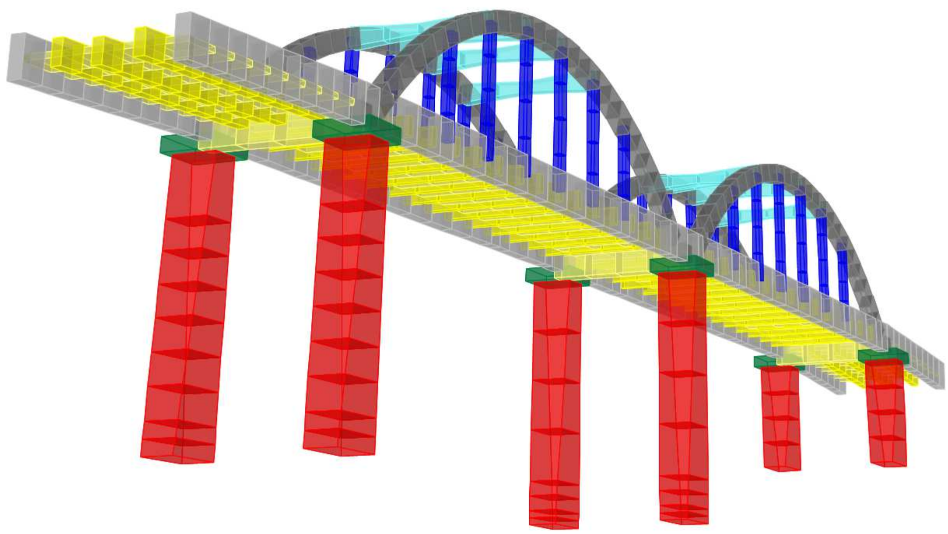
Limits of Pier Cap Retrofit shown above in Red



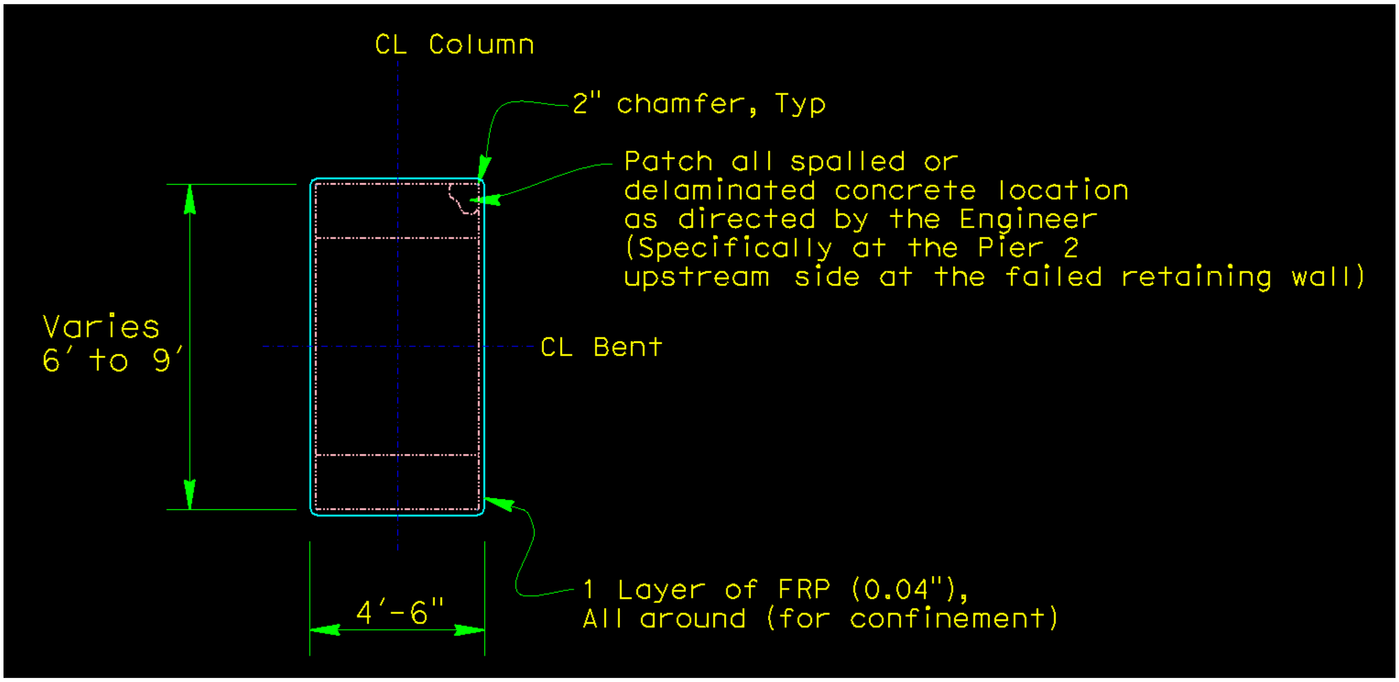
Section of Cap Retrofit

<u>Pier Retrofit</u>

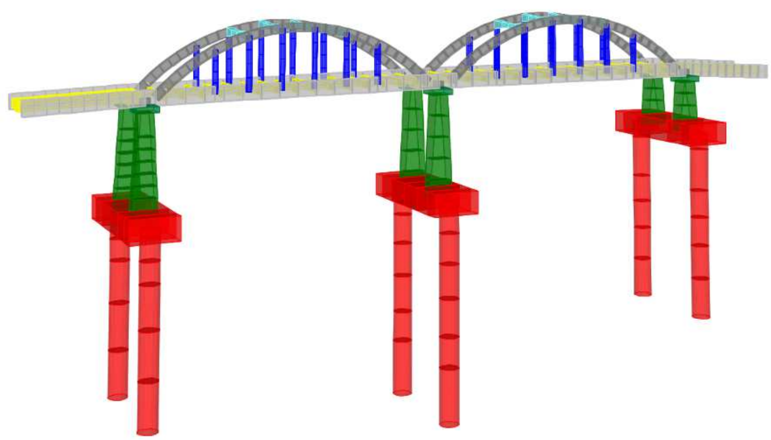
The team evaluated numerous models in determining the proposed retrofit strategy. In general, making the piers more flexible in the transverse direction increased the structure period and reduced loads in the superstructure (resulting is less retrofit of superstructure members). Therefore, it was determined to allow the piers to behave inelastically which requires fiber wrap to increase shear capacity and ductility. Another factor that reduces pier stiffness is the removal of the curtain walls between piers at Pier 2 and Pier 3. A curtain wall does not exist at Pier 4. The added benefit to removal of the curtain walls is it makes it easier to wrap all piers with FRP for confinement.



Limits of Pier Retrofit shown in Red



Pier Footing and Piles Retrofit

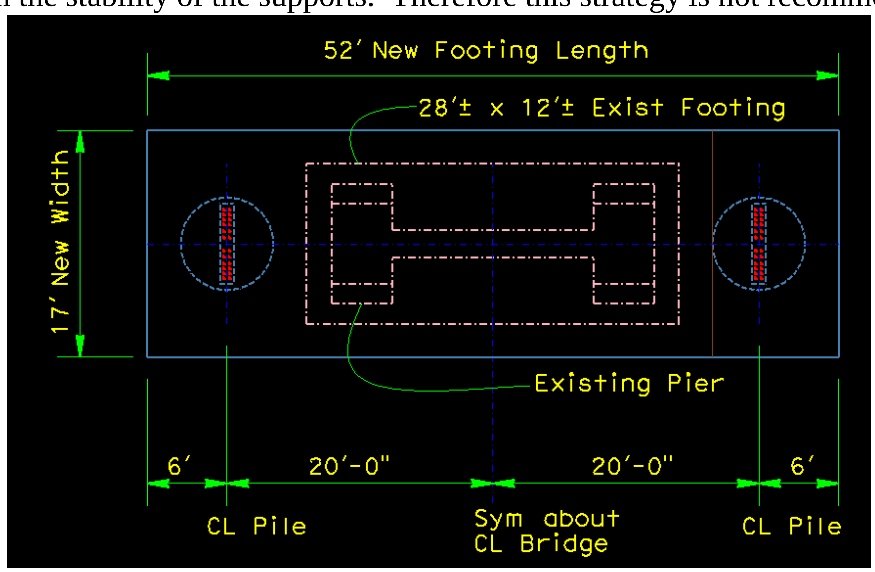


Limits of Pier Footing and Pier Piles shown in Red

The retrofit of the Pier Footing foundations are required to maintain stability under scour events. The scour issues discussed earlier are a significant threat to the bridge and the foundations must be strengthened to resist this condition. The foundations must also be able to withstand seismic demands. While these two conditions do not occur at the same time, the retrofit will account for both cases, with the more severe of the two conditions controlling the design. A detailed seismic analysis was not performed on the existing footing or piles since the scour demands required that they be retrofitted regardless of seismic performance.

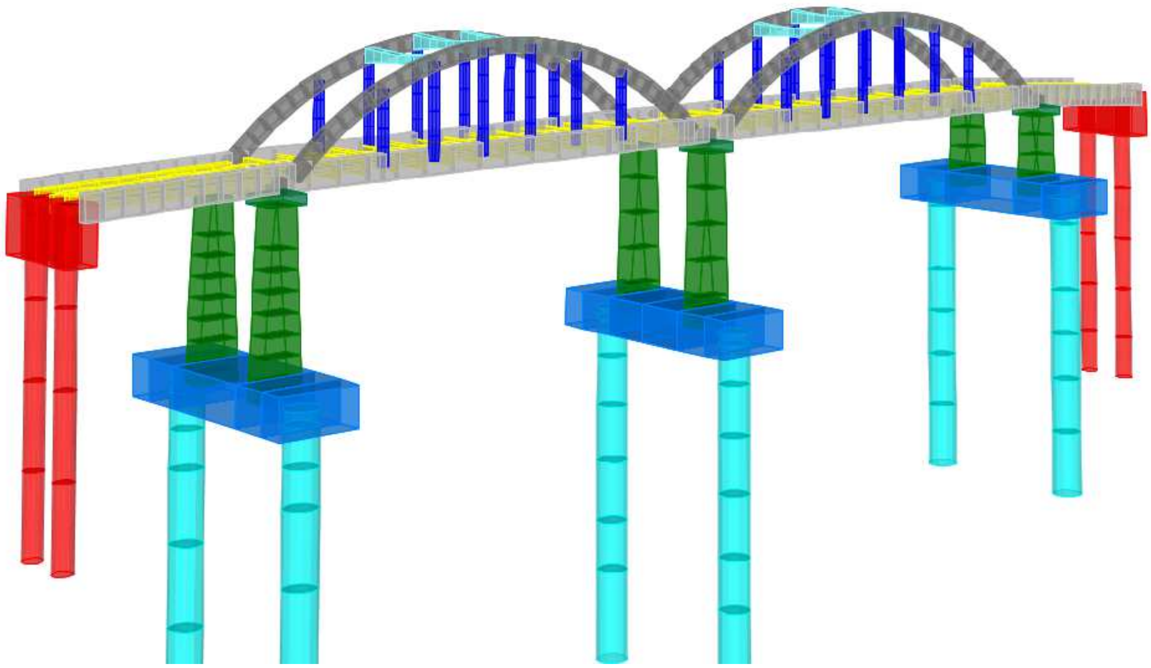
Deep foundations consisting of large diameter CIDH piles will be installed and connected to the existing Pier Footings. The piles will provide both vertical and lateral support to supplement and/or replace the existing foundations depending on the load condition. Each pier support will require two 84" large diameter piles placed outside of existing pier footing footprint. It is proposed to pin the top of the retrofit piles in the transverse direction, but keep them fixed in the longitudinal direction. In general demand loads in the superstructure decrease with a stiffer structure in the longitudinal direction yet a more flexible structure in the transverse direction. A transverse pin also eliminates the retrofit pile plastic moments from having to be resisted in the transverse direction at the pier footing.

Another strategy considered included separating the superstructure from the substructure by the means of "Base Isolation". Base Isolation is a strategy where bearings are installed between the superstructure and the substructure which effectively allows these two components to move independently. These bearings would decrease the seismic demands in the entire bridge by effectively lengthening the structural period, i.e., making the bridge more flexible. This strategy was not considered feasible due to a host of structural complications associated with disconnecting the arch spans from their supports. In addition, it would not address scour issues that threaten the stability of the supports. Therefore this strategy is not recommended.

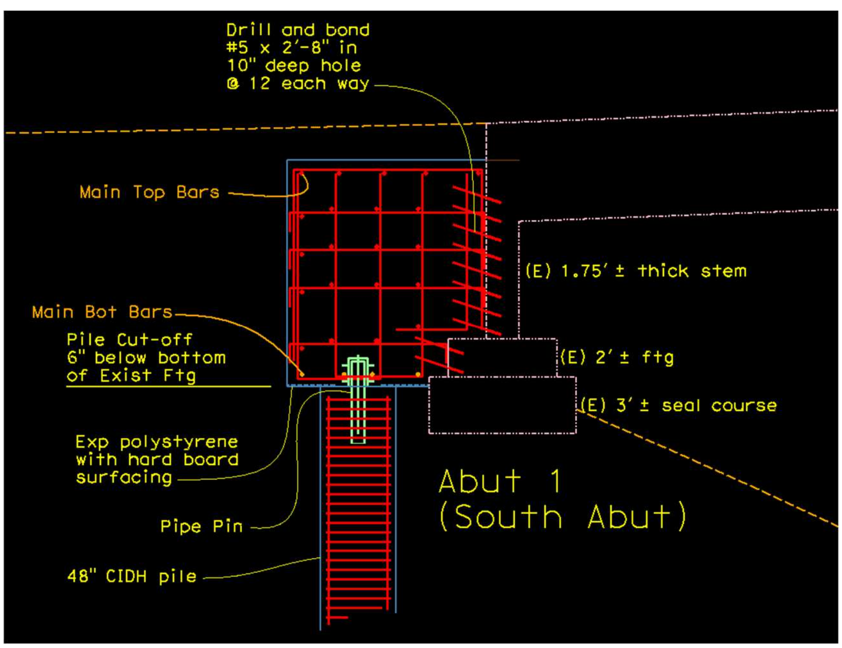


Abutment Retrofit

Abutment retrofit is required for two reasons. First, the structure needs to be stiffened in the longitudinal direction which reduces longitudinal displacement and seismic force demands in the superstructure. Secondly, settlement at the abutments needs to be stopped as the approach spans show large negative moment cracking near the pier which could indicate that the abutments have been settling over time. Due to these reasons, two 48" CIDH piles are recommended behind the existing abutments. Unfortunately, the abutment retrofit piles increase demands in the transverse direction for the Arch Ribs and Tie Girders. Therefore, the retrofit pile connection detail will be designed to be released in the transverse direction and fixed in the longitudinal direction.



Limits of Abutment Diaphragm Bolster and Abutment Piles shown in Red



Concrete Railing Repair/Replacement

The existing concrete railing is in poor condition. Several locations have spalled and have exposed steel reinforcement. What makes repair of the railing difficult is that the rail is considered a historical character defining feature of the bridge. Therefore, it must be repaired or replaced in-kind in order to prevent an adverse effect on the historical resource.

Preservation of the existing rail is preferred and replacement on the rails will only be considered if repair is unfeasible. This does result is some risk to the County because the existing railing is not crash tested and may not satisfy the latest crash test requirements. Typically, agencies are not required to upgrade their bridge rails if they are just making repairs to an existing railing. Agencies are required to upgrade their bridge rails for new or replacement projects. Therefore, it may be best to salvage portions of the existing railing that are still in fair condition. It is important to note that railing adjacent to the verticals must be removed to allow for fiber wrapping or replacement of the vertical members. Since there are so many vertical members very little of the existing railing will remain. Therefore, cost estimates have assumed replacement of the entire railing in kind.

10.CONSTRUCTION COSTS

Construction costs have been developed based on preliminary quantities and unit costs for similar projects. A 10% mobilization and 20% contingency are included in the total costs to account for uncertainty in the preliminary phase. Costs are presented in 2017 dollars. The estimated construction cost is $10,213,000 and is broken down in the following major categories below. A detailed individual quantity estimate is located in **Appendix B**.

Bridge Retrofit	$5,465,580
General Repairs	$1,014,850
Railing Repair/Replacement	$ 436,800
Rock Slope Protection	$ 215,560
Roadway	$ 664,895
Mobilization	$ 713,269
Contingency	$1,702,146
Total	$10,213,000

The project is currently programmed in the March 2017 Caltrans HBP FTIP list at $6,372,000 for the CON phase. Since the estimated cost is higher than the programmed amount it is recommended that the programming be increased. A 6B and 6D will also be necessary to finalize the increased funding programming. Please note that this construction costs does not included costs associated with the design, right of way and utility phases. This cost also does not include County administrative costs, or costs associated with construction engineering.

Following completion of the 95% design, the engineer's estimate will be updated utilizing final bridge design quantities and updated unit prices that reflect the most accurate historical cost information available at the time.

11.CONCLUSIONS AND RECOMMENDATIONS

The proposed retrofit strategy recommends Fiber Reinforced Polymer (FRP) to strengthen and confine the Arch Rib, Portal Bracing, and Vertical Hanger members so that they remain essentially elastic during a seismic event. Reinforced concrete bolsters are also proposed to strengthen the Tie Girders, Approach Span Exterior Girders, Pier Caps and Transverse Floor Beams adjacent to the Piers. The bolsters will allow these superstructure elements to remain elastic during a seismic event and resist plastic moments and shears imparted by the Piers. All members in the superstructure are lightly confined, therefore they have a very low ductility and failure occurs shortly after the member yields. The proposed retrofit with FRP adds the necessary confinement required for ductile behavior, but also increases the member strength. For all superstructure members, it became possible to keep members essentially elastic by applying minimal additional layers of FRP. Since most of the FRP costs will be associated with providing access and equipment to install FRP, the incremental cost to add additional layers is minimal. Therefore, it is proposed to provide enough FRP to keep members elastic. This approach reduces damage during an earthquake and increases the factor of safety at a minimal cost increase.

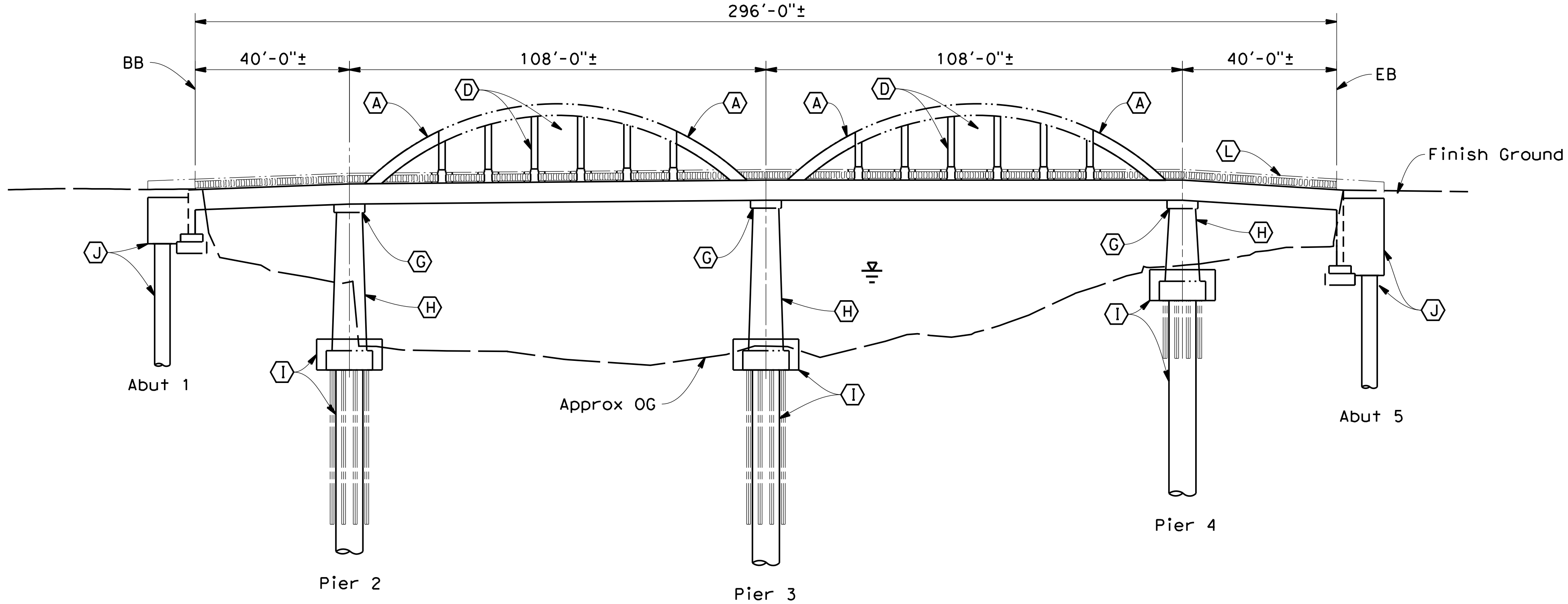
FRP is also proposed to confine all Piers. This will require the removal of the curtain walls between the piers where curtain walls exist (Pier 2 and Pier 3). Sensitivity analysis shows that the structure performs better in the transverse direction without the curtain walls. Therefore, it is proposed not to reconstruct them. Piers will be allowed to behave inelastically, however a push over analysis shows that the FRP will provide adequate confinement to accommodate demand displacements.

CIDH piles are proposed behind the abutments and at the piers. At the abutments the CIDH piles will resist scour, provide increased stiffness in the longitudinal direction for seismic loading (which reduces superstructure demands), and prevent future abutment settlement which appears to be occurring based on deck crack observations. At the piers the CIDH piles will support the footing under the scour condition and also resist seismic loading.

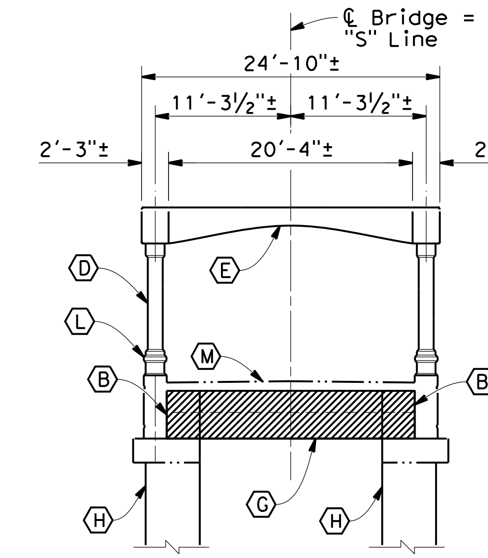
General repairs are also proposed to restore the bridge to its As-built condition. These repairs are summarized in greater detail in the Alta Vista report located in **Appendix G**. Repairs will include removal of existing unsound concrete, cleaning and painting of exposed reinforcing steel, and patching of spalled areas with new concrete. Alta Vista recommends repairs for 2,258 sqft of concrete surface area. This was comprised of approximately 775 sqft of deck area, 40 sqft of girder area, 1,406 sqft of soffit area, and 37 sqft of arch, portal, and vertical hanger area. Epoxy crack injection and methacrylate are also proposed on the deck to reduce water intrusion and extend the service life of the structure. Rock slope protection is proposed to protect both abutment slopes. Lastly the bridge rail will be repaired or replaced in kind.

Since environmental has been completed it is recommended that design proceed to the final PS&E phase after review and approval of this project report. Securing of temporary construction easements and possible utility relocations to provide for crane access should also be evaluated concurrently during this phase.

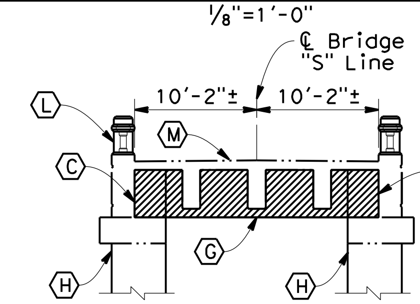
Appendix A - Structure Rehabilitation General Plan



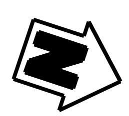
ELEVATION (K)
1"=20'

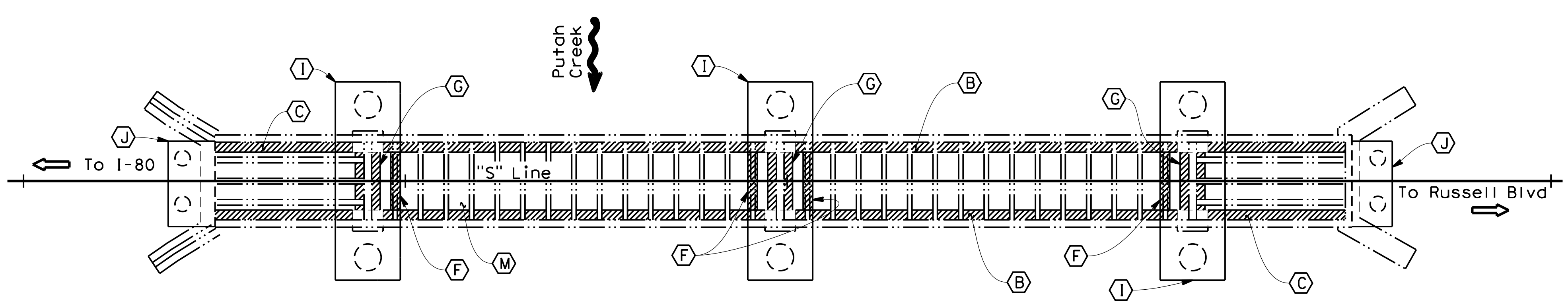


ARCH SPAN TYPICAL SECTION (K)
⅛"=1'-0"



APPROACH SPAN TYPICAL SECTION (K)
⅛"=1'-0"





PLAN (K)
1"=20'

RETROFIT LEGEND

- (A) Arch Rib - Fiber Wrap (Spring line to first vertical hanger)
- (B) Tie Girder - Bolster
- (C) Approach Span Exterior Girder - Bolster
- (D) Vertical Hanger - Fiber-Warp or Replace
- (E) Portal Bracing - Fiber Wrap
- (F) Floor Beam - Bolster
- (G) Pier Cap - Bolster
- (H) Pier Column - Remove Internal Pier Wall and Fiber Wrap Columns
- (I) 7' Diameter CIDH Pile and Pier Footing Buildout
- (J) 4' Diameter CIDH Pile and Abutment Diaphragm Bolster behind Existing Abutment
- (K) Remove Unsound Concrete and Patch Spalls
- (L) Concrete Railing - Remove and Reconstruct
- (M) Deck - Epoxy Inject Cracks and Methacrylate

LEGEND:

- Indicates Bolster
- Indicates New Construction
- Indicates Existing structure
- Indicates Highwater elevation
- Indicates Direction of Flow
- Indicates Direction of Traffic

REVISIONS NO.	DESCRIPTION	APPROVED BY	DATE

FIELD BOOK NO.

SCALE
HORIZONTAL: AS NOTED
VERTICAL: AS NOTED

DRAWN BY: CHECKED BY:
SUBMITTED R.C.E. No.

SOLANO COUNTY
TRANSPORTATION DEPARTMENT
333 SUNSET AVE. SUITE 230
SUISUN CITY CA 94585
TEL: (707) 421-6069 FAX: (707) 429-2894

APPROVED BY:
DATE:

PUTAH CREEK BRIDGE REHABILITATION
ON STEVENSON BRIDGE ROAD
GENERAL PLAN

DATE 1-4-2018
SHEET 1 OF 1
DWG

Appendix B - Quantities & Cost Estimate

Q uincy E ngineering, I nc.

GENERAL PLAN 20% CONTINGENCY

Project Name	Stevenson Road Bridge	Date	1/10/18
		Project. No.	S31-200
Bridge Name	Putah Creek Bridge (Retrofit and Rehabilitation)	Bridge Q's By	J. Chou
Bridge. No.	23C0092	Bridge Check Q's By	G. Young

Item No.		Item Code	Item Description	Unit	Quantity	Unit Price	Total
1		130600	TEMPORARY DIVERSION SYSTEM	LS	LUMP SUM	$ 100,000.00	$ 100,000.00
2	F	192003	STRUCTURE EXCAVATION (BRIDGE)	CY	742	$ 150.00	$ 111,300.00
3	F	192008	STRUCTURE EXCAVATION (TYPE A)	CY	760	$ 350.00	$ 266,000.00
4	F	193003	STRUCTURE BACKFILL (BRIDGE)	CY	600	$ 160.00	$ 96,000.00
5		480300	TEMPORARY SUPPORT	LS	LUMP SUM	$ 200,000.00	$ 200,000.00
6		490607	48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	320	$ 900.00	$ 288,000.00
7		490616	84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	660	$ 3,000.00	$ 1,980,000.00
8	F	510051	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	600	$ 650.00	$ 390,000.00
9	F	510053	STRUCTURAL CONCRETE, BRIDGE	CY	482	$ 1,600.00	$ 771,200.00
10		511106	DRILL AND BOND DOWEL	LF	1,642	$ 40.00	$ 65,680.00
11		511111	DRILL AND BOND DOWEL (CHEMICAL ADHESIVE) (LF)	LF	2,092	$ 55.00	$ 115,060.00
12	F	520102	BAR REINFORCING STEEL (BRIDGE)	LB	310,000	$ 1.50	$ 465,000.00
13		600003	INJECT CRACK (EPOXY)	LF	170	$ 60.00	$ 10,200.00
14		600011	RAPID SETTING CONCRETE (PATCH)	CF	775	$ 80.00	$ 62,000.00
15		600013	REPAIR SPALLED SURFACE AREA	SQFT	1,854	$ 440.00	$ 815,760.00
16		600014	FIBER-WRAP	SQFT	8,530	$ 60.00	$ 511,800.00
17		600033	REMOVE UNSOUND CONCRETE	CF	775	$ 120.00	$ 93,000.00
18		600037	PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	5,920	$ 4.00	$ 23,680.00
19	F	600045	TREAT BRIDGE DECK	SQFT	5,920	$ 1.00	$ 5,920.00
20		600047	FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	66	$ 65.00	$ 4,290.00
21		600068	CORE CONCRETE (6")	LF	156	$ 240.00	$ 37,440.00
22		600114	BRIDGE REMOVAL (PORTION)	LS	LUMP SUM	$ 50,000.00	$ 50,000.00
23		723060	ROCK SLOPE PROTECTION (300 lb, Class IV, METHOD B) (CY)	CY	800	$ 260.00	$ 208,000.00
24		729011	ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	945	$ 8.00	$ 7,560.00
25	F	750501	MISCELLANEOUS METAL (BRIDGE)	LB	1,200	$ 15.00	$ 18,000.00
26		839791	RECONSTRUCT CONCRETE RAILING (BRIDGE)	LF	672	$ 650.00	$ 436,800.00
27		999990	MOBILIZATION	LS	LUMP SUM	$ 713,269.00	$ 713,269.00
						SUBTOTAL CONTRACT	$ 7,845,959.00
		SUPPLEMENTAL WORK					
28						$ -	
29						$ -	
30						$ -	
31						$ -	
32						$ -	
						SUBTOTAL SUPPLEMENTAL WORK	$ -
						SUBTOTAL	$ 7,845,959.00
				CONTINGENCIES		20.0%	$ 1,569,041.00
				TOTAL			**$ 9,415,000.00**

Quincy Engineering, Inc.

PROJECT REPORT 20% CONTINGENCY

Project Name	Stevenson Road Bridge	Date	1/10/2018
Bridge Name	Roadway Improvements	Project. No.	S31-200
Bridge. No.	23C0092	Road Q's By	A. Mitchell
		Road Check Q's By	B. Road

Item No.		Item Code	Item Description	Unit	Quantity	Unit Price	Total
1		070030	LEAD COMPLIANCE PLAN	LS	LUMP SUM	$ 5,000.00	$ 5,000.00
2		120090	CONSTRUCTION AREA SIGNS	LS	LUMP SUM	$ 8,000.00	$ 8,000.00
3		120100	TRAFFIC CONTROL SYSTEM	LS	LUMP SUM	$ 15,000.00	$ 15,000.00
4		120120	TYPE III BARRICADE	EA	6	$ 200.00	$ 1,200.00
5		130100	JOB SITE MANAGEMENT	LS	LUMP SUM	$ 5,000.00	$ 5,000.00
6		130300	PREPARE STORM WATER POLLUTION PREVENTION PLAN	LS	LUMP SUM	$ 2,500.00	$ 2,500.00
7		130310	RAIN EVENT ACTION PLAN	EA	5	$ 500.00	$ 2,500.00
8		130320	STORM WATER SAMPLING AND ANALYSIS DAY	EA	4	$ 1,500.00	$ 6,000.00
9		130330	STORM WATER ANNUAL REPORT	EA	1	$ 2,000.00	$ 2,000.00
10		130640	TEMPORARY FIBER ROLL	LF	2150	$ 4.00	$ 8,600.00
11		130680	TEMPORARY SILT FENCE	LF	2150	$ 5.00	$ 10,750.00
12		130710	TEMPORARY CONSTRUCTION ENTRANCE	EA	2	$ 1,000.00	$ 2,000.00
13		130900	TEMPORARY CONCRETE WASHOUT	LS	LUMP SUM	$ 2,000.00	$ 2,000.00
14		131103	WATER QUALITY SAMPLING AND ANALYSIS DAY	EA	8	$ 500.00	$ 4,000.00
15		131104	WATER QUALITY MONITORING REPORT	EA	4	$ 500.00	$ 2,000.00
16		170103	CLEARING AND GRUBBING (LS)	LS	LUMP SUM	$ 25,000.00	$ 25,000.00
17		190101	ROADWAY EXCAVATION	CY	880	$ 50.00	$ 44,000.00
18		198010	IMPORTED BORROW (CY)	CY	160	$ 80.00	$ 12,800.00
19		210252	BONDED FIBER MATRIX (SQFT)	SQFT	24450	$ 1.00	$ 24,450.00
20		260203	CLASS 2 AGGREGATE BASE (CY)	CY	1910	$ 100.00	$ 191,000.00
21		390132	HOT MIX ASPHALT (TYPE A)	TON	1030	$ 200.00	$ 206,000.00
22		397005	TACK COAT	TON	0.5	$ 1,300.00	$ 650.00
23		839543	TRANSITION RAILING (TYPE WB-31)	EA	4	$ 3,000.00	$ 12,000.00
24		839584	ALTERNATIVE IN-LINE TERMINAL SYSTEM	EA	4	$ 3,000.00	$ 12,000.00
25		999990	MOBILIZATION	LS	LUMP SUM	$ 60,445.00	$ 60,445.00
			SUBTOTAL CONTRACT				$ 664,895.00
		SUPPLEMENTAL WORK					
26						$ -	
27						$ -	
28						$ -	
29						$ -	
30						$ -	
			SUBTOTAL SUPPLEMENTAL WORK				$ -
				SUBTOTAL			$ 664,895.00
				CONTINGENCIES		20.0%	$ 133,105.00
				TOTAL			**$ 798,000.00**

Stevenson Bridge Retrofit
Quantities & Estimate
10-12-2017

Table of Contents

Temporary Diversion System [LS]

Temporary Diversion System may be required to install abutment RSP systems, Temporary Support, and other construction elated activities.

Past project estimates and contractor bid prices are listed below:

Del Norte County, Hurdy-gurdy, Temporary Stream Diversion, LS, $200k (2017 bid)

Harbin Spring Road, Harbin Creek Bridge, Temporary Diversion System, $47k (2017 bid)

Lake County, Dry Creek Road Bridge, Temporary Stream Diversion, $150k (2016 bid)

Lake County, Foard Road Bridge, Temporary Stream Diversion, $70k (2016 bid)

Santa Barbara County, Goleta Slough Bridge, Temporary Stream Diversion, $75k (2016 bid)

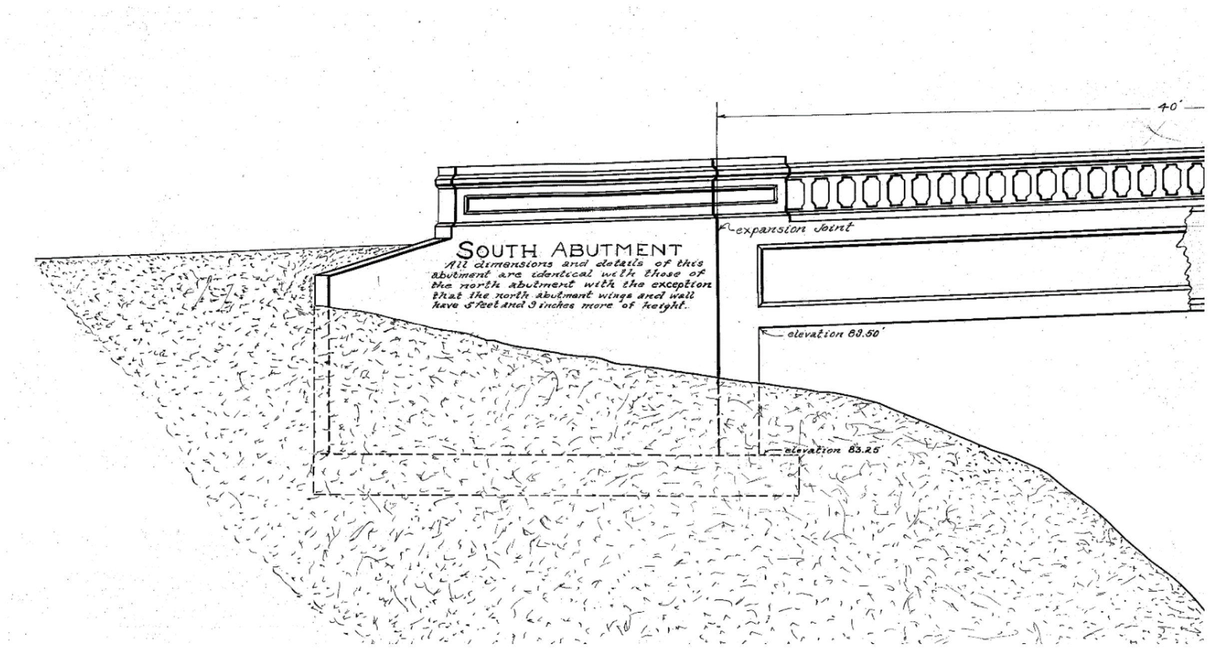
Trinity County, Halls Gulch Bridge, Trinity River Diversion, LS, $60k (2013 estimate)

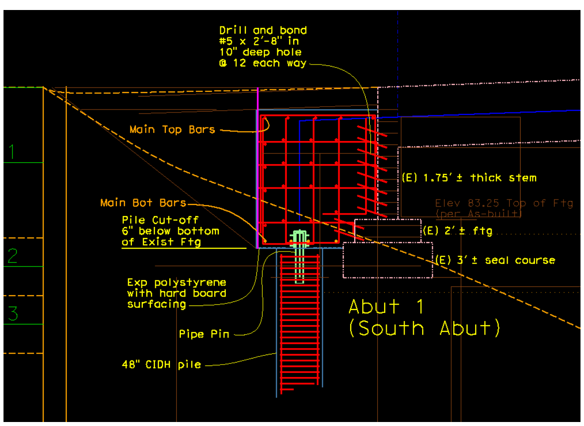
Stevenson Estimated price = $\boxed{\$100,000/LS}$

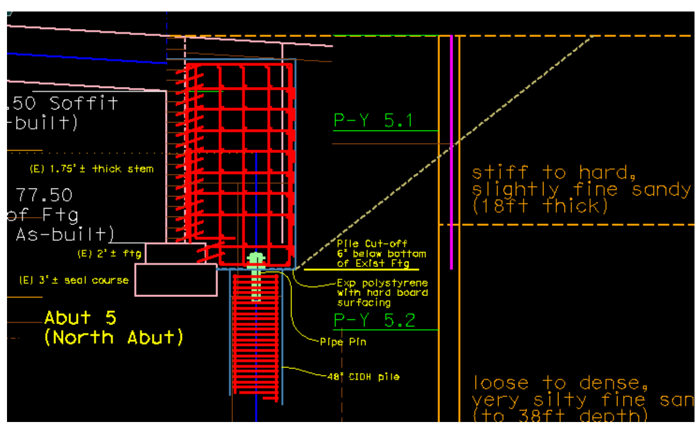
Structure Excavation (Bridge) [CY]

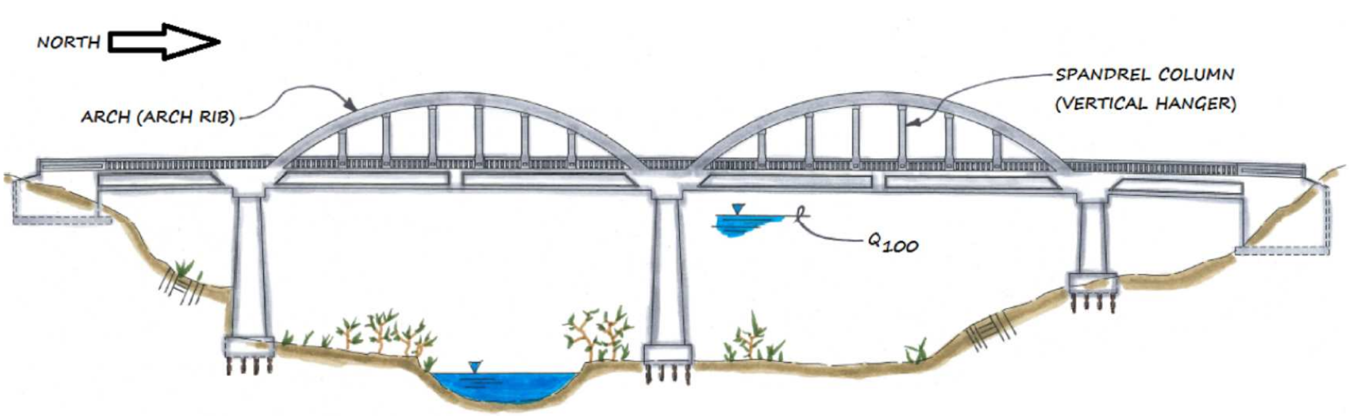
Abutment 1 location:

Structure Excavation (Bridge) – Continued









Structure Excavation (Bridge) – Continued

Due to the proximity of the creek, Abutment 1, Pier 4 and Abutment 5 are Structure Excavation Bridge. (Pier 2 and Pier 3 are Structure Excavation Type A.)

Abut 1: [(14' tall from deck to pile cut-off Abut 1 footing) (22' wide at Abut 1 face) (12' length, longitudinally)] / 27

= 137 CY

Pier 4: [(average ~10' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27

= 380 CY

Abut 5: [(23' tall from deck to pile cut-off at Abut 5 footing) (22' wide at Abut 1 face) (12' length, longitudinally)] / 27

= 225 CY

∑ = 742 CY

Say $\boxed{\text{742CY}}$

Based on Caltrans bid history, the average adjusted Structure Excavation (Bridge) is around \$105/CY. The average adjusted Structure Excavation (Type A) is around \$350/CY. (See next page.) For no seal course anticipated at Abutment 1, Pier 4 and Abut 5 locations, use the structure excavation bridge unit price of \$200/CY for Stevenson Bridge.

Estimated price = $\boxed{\text{\$150/CY}}$

In 2007, the estimated Stevenson Bridge unit price for Structure Excavation (Bridge) was \$150/CY.

Structure Excavation (Bridge) – Continued

☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	04	772	$45.00	$45.19
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	04	772	$225.00	$225.93
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	04	772	$70.00	$70.29
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	04	772	$270.00	$271.12
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	04	772	$125.00	$125.52
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	06	959	$20.67	$20.67
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	06	959	$22.00	$22.00
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	06	959	$65.00	$65.00
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	06	959	$100.00	$100.00
☑	192003 - STRUCTURE EXCAVATION (BRIDGE)	CY	06	959	$110.00	$110.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	104.35	**187.77**	Avg No. Units	854
Std Dev. (of Unit Price): ±$	114.09	**201.29**	Rows Selected	298
Weighted Avg.: $	103.43	**185.17**	Rows Returned	298
Minimum Price/Unit: $	13.00	17.03		
Maximum Price/Unit: $	1,000.00	1,593.53		

☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$190.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$110.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$400.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$130.00

uncheck all | check all

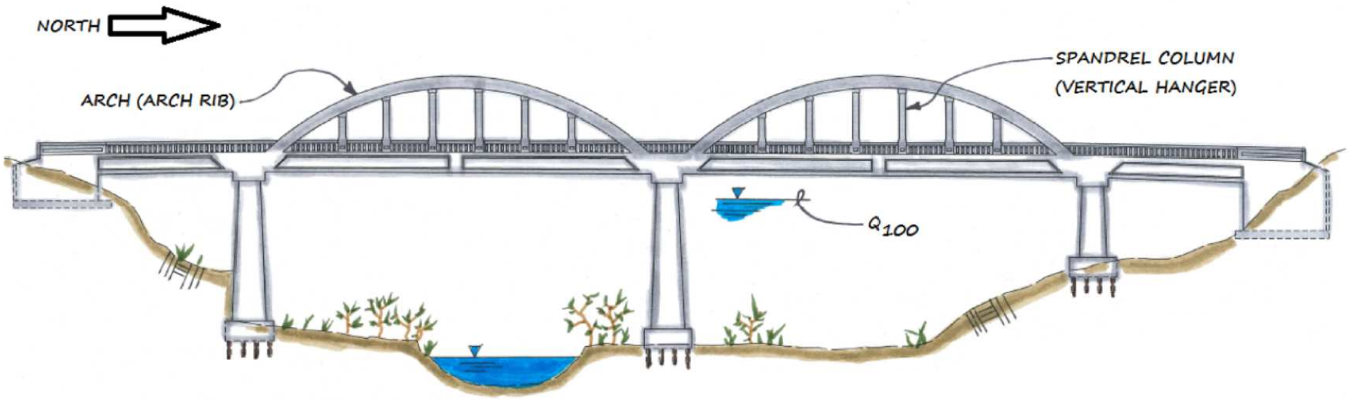
SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	292.41	**344.18**	Avg No. Units	2242
Std Dev. (of Unit Price): ±$	266.20	**326.02**	Rows Selected	79
Weighted Avg.: $	292.92	**341.14**	Rows Returned	79
Minimum Price/Unit: $	10.00	17.60		
Maximum Price/Unit: $	1,529.11	2,280.71		

☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$50.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$150.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$75.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$200.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	123.79	**127.60**	Avg No. Units	2375
Std Dev. (of Unit Price): ±$	157.46	**152.33**	Rows Selected	190
Weighted Avg.: $	121.39	**125.73**	Rows Returned	190
Minimum Price/Unit: $	4.59	6.48		
Maximum Price/Unit: $	1,529.11	1,163.35		

Structure Excavation (Type A) [CY]



Pier 2 location:

Structure Excavation (Type A) – Continued

Due to the proximity of the creek, Pier 2 and Pier 3 are Structure Excavation Type A. (Abutment 1, Pier 4 and Abutment 5 are Structure Excavation Bridge.)

Pier 2: [(average ~15' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27

= 570 CY

Pier 3: [(about 5' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27

= 190 CY

$\sum = 760$ CY

Say $\boxed{760 \text{ CY}}$

Based on Caltrans bid history, the average adjusted Structure Excavation (Type D) is around $125/CY. The average adjusted Structure Excavation (Type A) is around $350/CY. (See next page.) Accounting for the possibility of seal course required at Pier 2 and Pier 3 locations, use the Type A price of $350/CY for Stevenson Bridge.

Estimated price = $\boxed{\$350/\text{CY}}$

In 2007, the estimated Stevenson Bridge unit price for Structure Excavation (Bridge) was $150/CY.

Structure Excavation (Type A) – Continued

☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$190.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$110.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$400.00
☑	192008 - STRUCTURE EXCAVATION (TYPE A)	CY	04	4279	$130.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	292.41	**344.18**	Avg No. Units	2242
Std Dev. (of Unit Price): ±$	266.20	**326.02**	Rows Selected	79
Weighted Avg.: $	292.92	**341.14**	Rows Returned	79
Minimum Price/Unit: $	10.00	17.60		
Maximum Price/Unit: $	1,529.11	2,280.71		

☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$50.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$150.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$75.00
☑	192020 - STRUCTURE EXCAVATION (TYPE D)	CY	04	1010	$200.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	123.79	**127.60**	Avg No. Units	2375
Std Dev. (of Unit Price): ±$	157.46	**152.33**	Rows Selected	190
Weighted Avg.: $	121.39	**125.73**	Rows Returned	190
Minimum Price/Unit: $	4.59	6.48		
Maximum Price/Unit: $	1,529.11	1,163.35		

Structure Backfill (Bridge) [CY]

Abut 1: [(14' tall from deck to pile cut-off Abut 1 footing) (22' wide at Abut 1 face) (1' length, longitudinally)] / 27

= 11 CY

Pier 2: [(average ~15' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27 – (8' Bottom Footing to FG) (52') (17) / 27

= 308 CY

Pier 3: [(5' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27 – (1' Bottom Footing to FG) (52') (17) / 27

= 157 CY

Pier 4: [(average ~10' high) (52' long + 1' +1') (17' wide + 1' + 1')] / 27 – (8' Bottom Footing to FG) (52') (17) / 27

= 118 CY

Abut 5: [(23' tall from deck to pile cut-off at Abut 5 footing) (22' wide at Abut 1 face) (1' length, longitudinally)] / 27

= 19 CY

∑ = 587 CY

Say $\boxed{\text{600 CY}}$

Based on Caltrans bid history, the average adjusted Structure Backfill (Bridge) is around \$100/CY. Use price of \$160/CY for Stevenson Bridge accounting for remote location.

☑	193003 - STRUCTURE BACKFILL (BRIDGE)	CY	08	697	$45.87
☑	193003 - STRUCTURE BACKFILL (BRIDGE)	CY	08	697	$56.58
☑	193003 - STRUCTURE BACKFILL (BRIDGE)	CY	08	697	$87.92
☑	193003 - STRUCTURE BACKFILL (BRIDGE)	CY	08	697	$38.23
☑	193003 - STRUCTURE BACKFILL (BRIDGE)	CY	08	697	$95.57

MORE THAN 500 RESULTS RETURNED. ONLY

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	71.66	**89.85**	Avg No. Units	560
Std Dev. (of Unit Price): ±$	51.18	**65.62**	Rows Selected	500
Weighted Avg.: $	69.18	**85.06**	Rows Returned	500
Minimum Price/Unit: $	5.00	8.68		
Maximum Price/Unit: $	500.00	868.42		

Estimated price = $\boxed{\text{\$160/CY}}$

In 2007, the estimated Stevenson Bridge unit price for Structure Backfill (Bridge) was \$120/CY.

Temporary Support [LS]

The Temporary Support item is necessary for the falsework necessary to support the cast in place reinforced concrete bolster work. The bolster is located under the bridge along the inside face of the exterior tie girder. The Temporary Support system is up to the Contractor's methods and means.

It is estimated to include hanger rods, timber form work, brackets, clamps, strips, ties, etc. The contractor may elect to support temporary falsework from the existing piers or may elect to support some falsework from the ground.

Estimated price = $\boxed{\$200{,}000\ \text{LS}}$

<u>48" Cast-in-Drilled-Hole Concrete Piling [LF]</u>

At this preliminary stage, either cast-in-drilled hole (CIDH) concrete piles or driven piles could potentially be used. However driven piles may not be as economical compared to CIDH concrete piles due to the high mobilization cost relative to the number of piles needed and the large construction footprint required to drive piles. Therefore, CIDH piles are proposed.

Abut 1 and Abut 5: (estimated 80' long per pile based on seismic analysis, without Geotech's vertical load analysis) (2 piles per support) (2 supports) = 320'

Say $\boxed{\text{320 LF}}$

☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	246	$550.00	$552.28	$135300.00	11-29-2016	04-209504	8	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	246	$725.00	$728.01	$178350.00	11-29-2016	04-209504	9	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$900.00	$900.00	$237600.00	01-19-2017	06-471504	1	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$640.00	$640.00	$168960.00	01-19-2017	06-471504	2	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$600.00	$600.00	$158400.00	01-19-2017	06-471504	3	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$810.00	$810.00	$213840.00	01-19-2017	06-471504	4	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$621.00	$621.00	$163944.00	01-19-2017	06-471504	5	M	TRO
☑	490607 - 48" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	06	264	$500.00	$500.00	$132000.00	01-19-2017	06-471504	6	M	TRO

uncheck all | check all

cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	505.69	**930.88**	Avg No. Units	265
Std Dev. (of Unit Price): ±$	272.77	**670.60**	Rows Selected	76
Weighted Avg.: $	509.84	**906.64**	Rows Returned	76

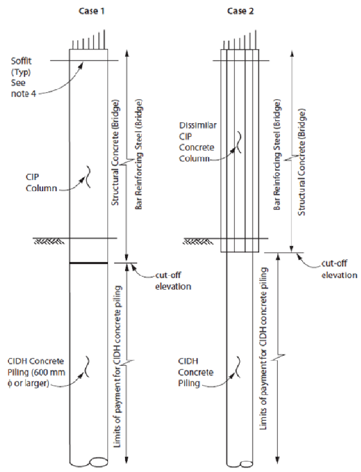
Based on Caltrans Contract Cost Data,

48" CIDH piles has an adjusted average price of about $900 for quantities between 100 to 600 LF. Access behind the abutment will be standard so the unit cost should not need to be increased for this factor.

Estimated price = $\boxed{\text{\$900/LF}}$

In 2007, the estimated Stevenson Bridge unit price for 60" Cast-in-Drilled-Hole Concrete Piling behind the abutment piles was $900/LF, the estimated quantity was 200LF which totals to $180k.

Pile Extensions and Columns for CIDH Concrete Piles



84" Cast-in-Drilled-Hole Concrete Piling [LF]

Given the necessary diameter cast-in-drilled hole (CIDH) concrete piles are proposed. CIDH piles are more cost effective than cast-in-steel-shell piles (CISS) which don't appear to be necessary given the seismic loading.

Pier 2, 3 and 4: (110' long per pile) (2 piles per support) (3 supports) = 660'

Say $\boxed{\text{660 LF}}$

☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$2180.00	$2476.51	$1318900.00	03-03-2016	08-0Q3004	6	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$2900.00	$3294.43	$1754500.00	03-03-2016	08-0Q3004	7	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$1500.00	$1704.02	$907500.00	03-03-2016	08-0Q3004	8	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$1850.00	$2101.62	$1119250.00	03-03-2016	08-0Q3004	9	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$1115.00	$1266.65	$674575.00	03-03-2016	08-0Q3004	10	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	08	605	$1600.00	$1817.62	$968000.00	03-03-2016	08-0Q3004	11	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$1700.00	$1707.05	$1011500.00	12-14-2016	04-235654	1	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$1600.00	$1606.63	$952000.00	12-14-2016	04-235654	2	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$1580.00	$1586.55	$940100.00	12-14-2016	04-235654	3	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$1200.00	$1204.97	$714000.00	12-14-2016	04-235654	4	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$2550.00	$2560.57	$1517250.00	12-14-2016	04-235654	5	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$1450.00	$1456.01	$862750.00	12-14-2016	04-235654	6	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$2359.14	$2368.92	$1403688.30	12-14-2016	04-235654	7	M	TRO
☑	490616 - 84" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	04	595	$2700.00	$2711.19	$1606500.00	12-14-2016	04-235654	8	M	TRO

uncheck all | check all   cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	1,444.96	**1,836.45**	Avg No. Units	662
Std Dev. (of Unit Price): ±$	611.60	**487.79**	Rows Selected	32
Weighted Avg.: $	1,380.76	**1,807.61**	Rows Returned	32
Minimum Price/Unit: $	650.00	1,204.97		
Maximum Price/Unit: $	2,900.00	3,294.43		

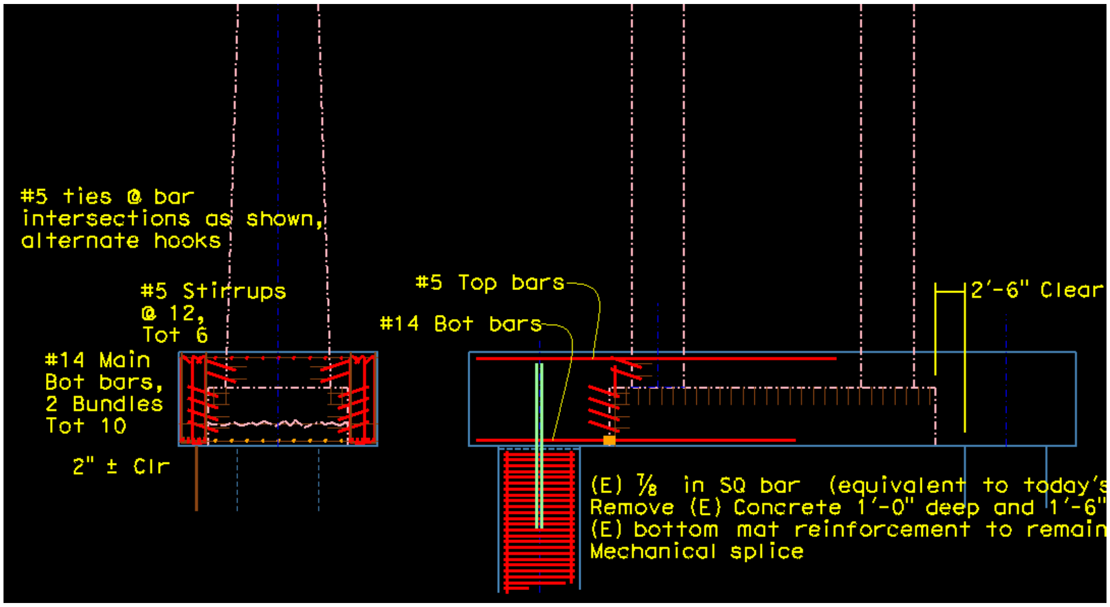
Based on Caltrans Contract Cost Data,

84" CIDH piles has an adjusted average price of about $1,800 for quantities between 500 to 1000 LF. Accounting for very difficult access and wet conditions an increase to the unit price is warranted.

Estimated price = $\boxed{\text{\$ 3,000/LF}}$

In 2007, the estimated Stevenson Bridge unit price for 84" Cast-in-Drilled-Hole Concrete Piling was $2,800/LF, the estimated quantity was 570LF which totals to $1,596k.

Structure Concrete, Bridge Footing [CY]



Pier 2: [(8' high) (52' long) (17' wide) – (5' high) (28' long) (12' wide)] / 27

= 200 CY

Pier 3: [(8' high) (52' long) (17' wide) – (5' high) (28' long) (12' wide)] / 27

= 200 CY

Pier 4: [(8' high) (52' long) (17' wide) – (5' high) (28' long) (12' wide)] / 27

= 200 CY

∑ = 600 CY

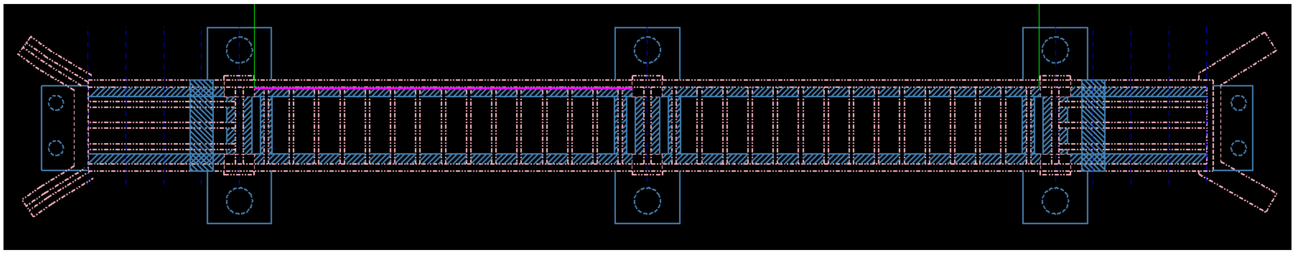
Say 600 CY

Based on Caltrans Contract Cost Data,

Structure Concrete, Bridge Footing has an average adjusted unit price of $500/CY--see next page.

Given the more difficult access assume a unit price of $650/CY

Structure Concrete, Bridge Footing – Continued



☑	510051 - STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	04	753	$229.37	$342.11
☑	510051 - STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	04	753	$229.37	$342.11
☑	510051 - STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	04	753	$382.28	$570.18
☑	510051 - STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	07	51	$191.14	$285.09

MORE THAN 500 RESULTS RETURNED. ONLY 500 ROWS SHOWN.

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	308.59	**498.71**	Avg No. Units	255
Std Dev. (of Unit Price): ±$	172.71	**260.68**	Rows Selected	500
Weighted Avg.: $	281.81	**459.76**	Rows Returned	500
Minimum Price/Unit: $	27.00	44.92		
Maximum Price/Unit: $	1,911.39	2,850.88		

In 2007, the estimated Stevenson Bridge unit price for 540 CY of Structure Concrete, Bridge was $1,300/CY. The estimated cost was $702k. The 2007 estimate did not have a Structural Concrete Footing item.

Structure Concrete, Bridge [CY]

Abut 1: [(12’ tall from deck to bottom of Abut 1 footing) (20’ wide at Abut 1 face) (10’ length, longitudinally)] / 27

= 89 CY

Abut 5: [(22’ tall from deck to bottom of Abut 5 footing) (20’ wide at Abut 1 face) (10’ length, longitudinally)] / 27

= 163 CY

Tie Girder Bolter: [(100’ long per quadrant – 1.167' x 15 floor beams) (31 / 12 ft wide) (47.6 / 12 ft tall) (4 quadrant) / 27 +

+ [(1.167' x 15 floor beams) (31 / 12 ft wide) (26 / 12 ft tall) (4 quadrant) / 27

= 140 CY

Approach Span Exterior Girder Bolter: [(36’ long per quadrant) (31 / 12 ft wide) (47.6 / 12 ft tall) (4 quadrant) / 27

= 55 CY

Pier 2,3,4 Bolster: [(15.167’ long per side) (2.25 ft wide) (4ft tall) (2 sides per support) (3 pier supports) / 27

= 30 CY

Floor beam Bolter adjacent to Arch: [(15.167’ long per side) (1 ft wide) (4ft tall) (2 sides per floor beam) (4 sets of floor beams) / 27 +

+ [(15.167’ long per side) (1.1667 ft wide) (2.1667ft tall $_{\text{below the existing floor beam}}$) (1 location per floor beam) (4 sets of floor beams) / 27

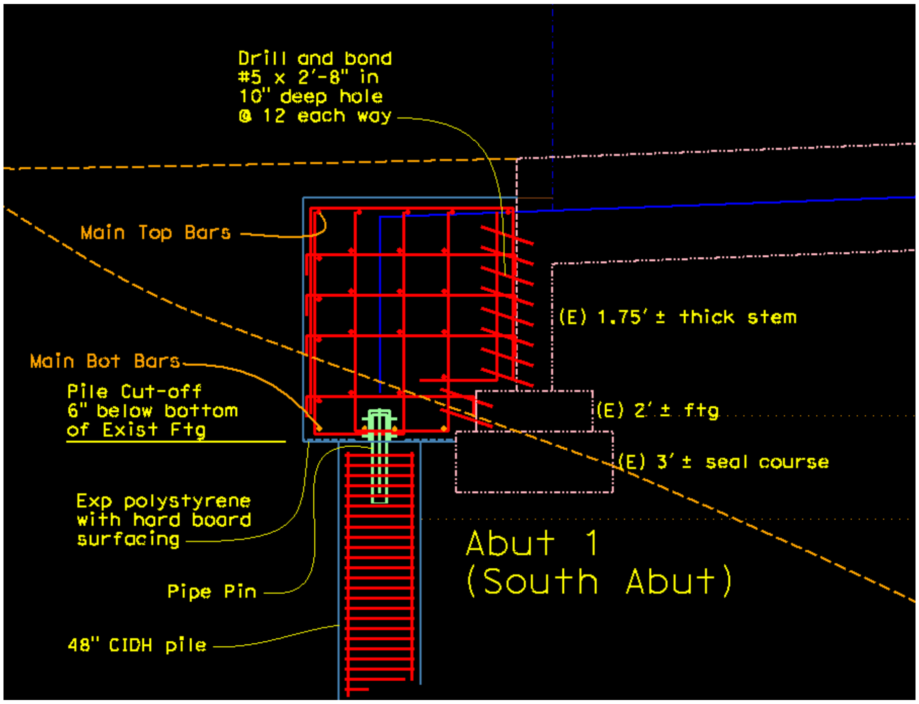
= 6 CY

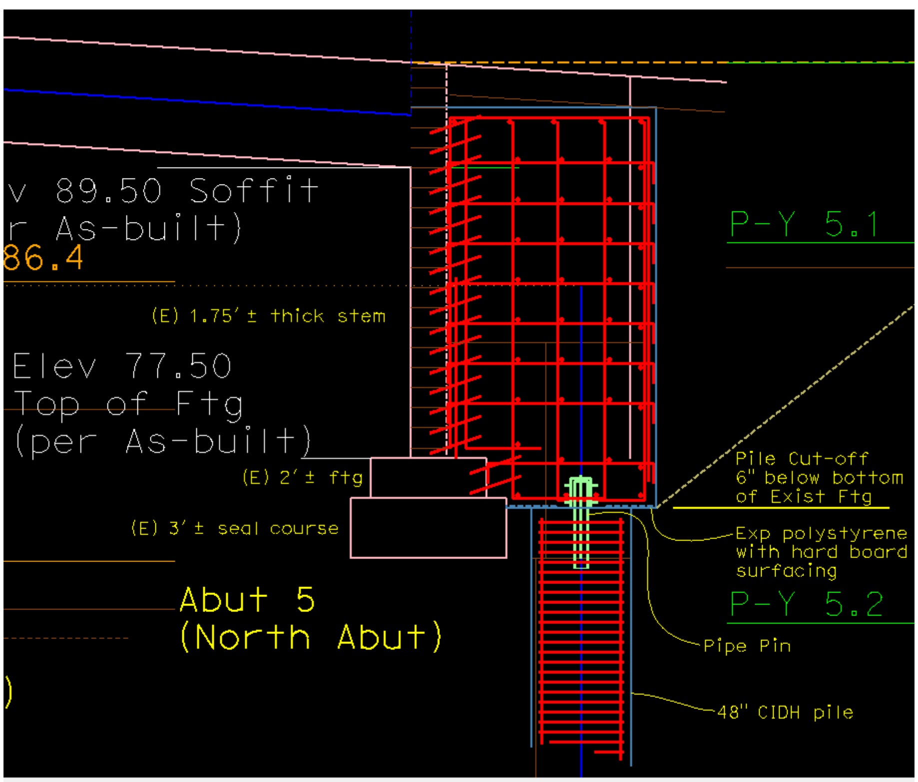
∑ = 482 CY

Say $\boxed{\text{482 CY}}$

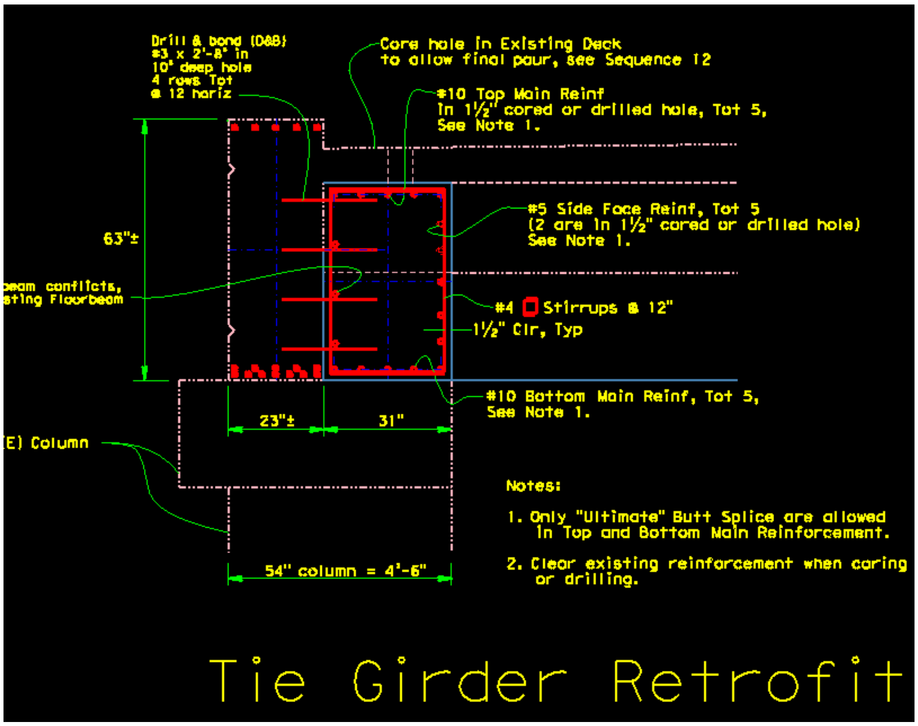
Based on Caltrans Contract Cost Data, the unit prices runs between 1,200 to 1,600/CY.
Structure Concrete, Bridge with unique a difficult retrofit for formwork and access,
say $\boxed{\text{\$1,600/CY}}$

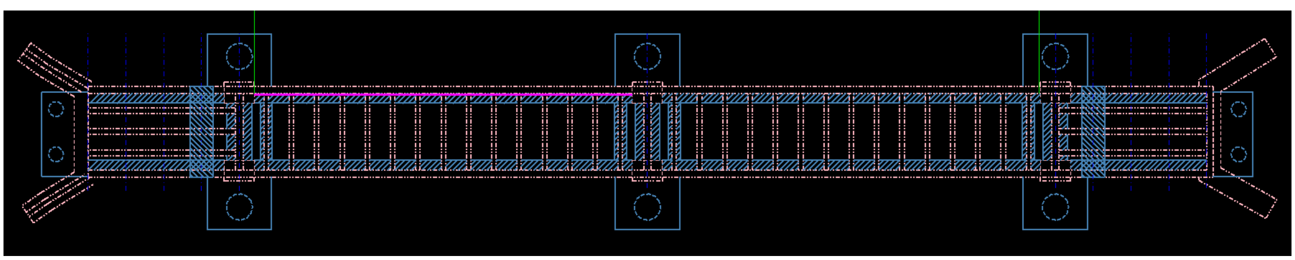
Structure Concrete – Continued



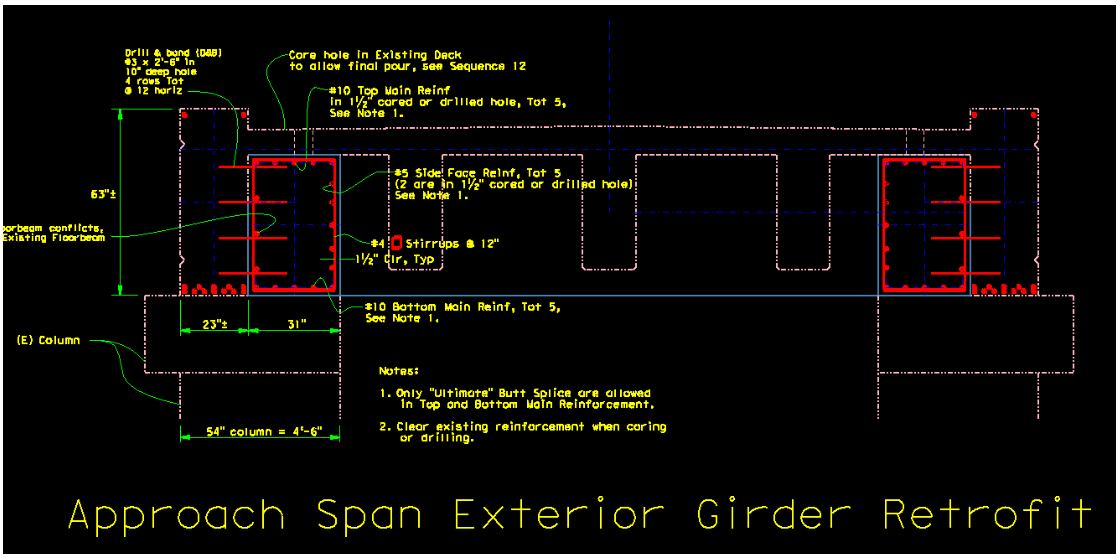


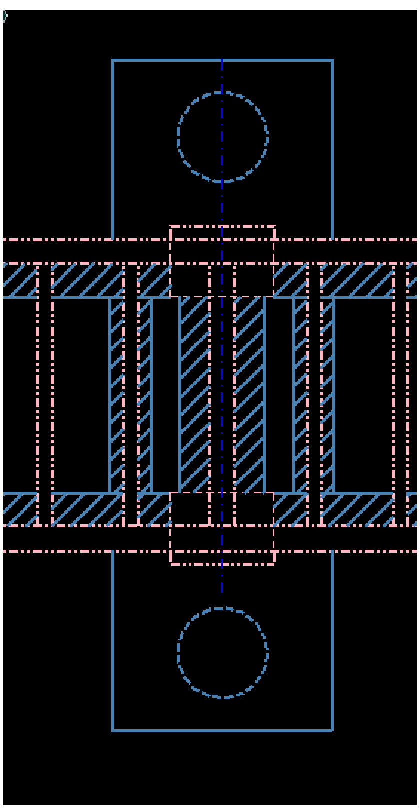
Structure Concrete, Bridge – Continued

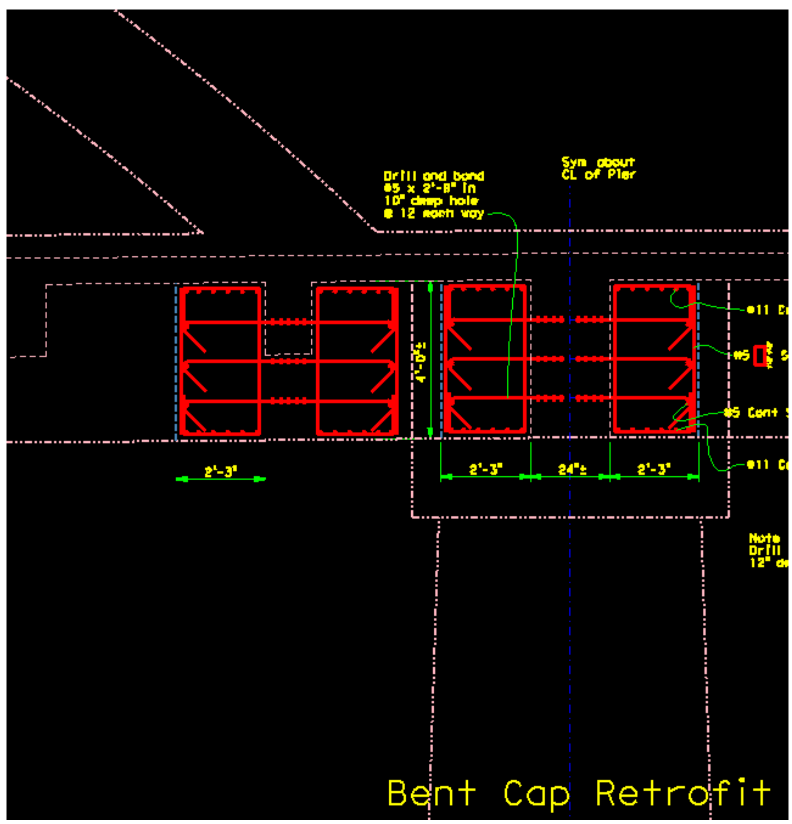




Structure Concrete, Bridge – Continued







Structure Concrete, Bridge – Continued

☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	01	545	$841.01	$891.50	$458700.00	04-18-2007	01-293144	1	M	
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	01	545	$917.47	$972.54	$500400.00	04-18-2007	01-293144	2	M	
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	01	545	$1299.74	$1377.77	$708900.00	04-18-2007	01-293144	3	M	
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	07	810	$1911.39	$2026.13	$1547500.00	05-10-2007	07-183114	1	M	TRO
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	07	810	$917.47	$972.54	$742800.00	05-10-2007	07-183114	2	M	TRO
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	04	854	$917.47	$972.54	$783600.00	05-30-2007	04-226144	1	M	TRO
☑	510053 - STRUCTURAL CONCRETE, BRIDGE	CY	04	854	$764.55	$810.45	$653000.00	05-30-2007	04-226144	2	M	TRO
MORE THAN 500 RESULTS RETURNED. ONLY 500 ROWS SHOWN.												

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cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	722.95	**1,628.04**	Avg No. Units	545
Std Dev. (of Unit Price): ±$	409.80	**718.57**	Rows Selected	500
Weighted Avg.: $	699.06	**1,567.17**	Rows Returned	500
Minimum Price/Unit: $	185.00	453.06		
Maximum Price/Unit: $	2,561.26	5,231.30		

In 2007, the estimated Stevenson Bridge unit price for 540 CY of Structure Concrete, Bridge was $1,300/CY. The estimated cost was $702k. The 2007 estimate did not have a Structural Concrete Footing item.

Drill and Bond Dowel [LF]

Drill and Bond Dowel includes the abutment drill and bond dowels that are installed at a 3:1 slope into the existing abutment stem.

Abut 1: [(8 Rows + 2 Rows to install for the 12' tall from deck to bottom of Abut 1 footing while leaving some cover for roadway section) (22 Columns to install over the 20' wide at Abut 1 face) (10/12 LF each)

= 183 LF

Abut 5: [(17 Rows + 2 Rows to install for the 12' tall from deck to bottom of Abut 1 footing while leaving some cover for roadway section) (22 Columns to install over the 20' wide at Abut 1 face) (10/12 LF each)

= 348.3 LF

Pier 2: [(6 Rows to install width of existing footing) (9 Columns to install width of existing footing) (2 sides) + (6 Rows to install width of existing footing) (28 Columns to install width of existing footing)] (2 sides) (10/12 LF each)

= 370 LF

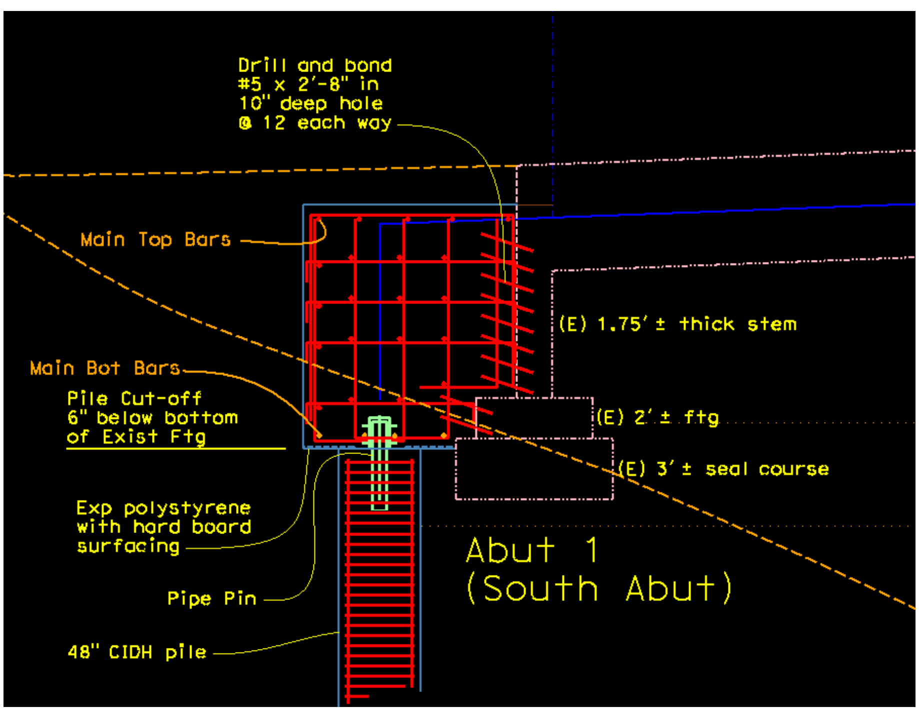
Pier 3: [(6 Rows to install width of existing footing) (9 Columns to install width of existing footing) (2 sides) + (6 Rows to install width of existing footing) (28 Columns to install width of existing footing)] (2 sides) (10/12 LF each)

= 370 LF

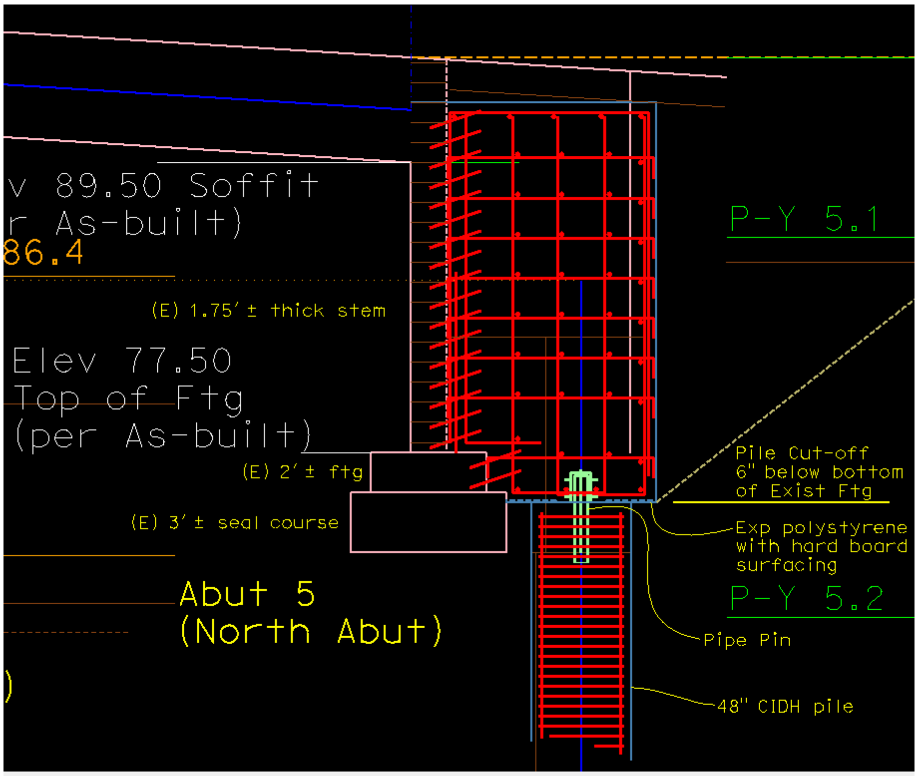
Pier 4: [(6 Rows to install width of existing footing) (9 Columns to install width of existing footing) (2 sides) + (6 Rows to install width of existing footing) (28 Columns to install width of existing footing)] (2 sides) (10/12 LF each)

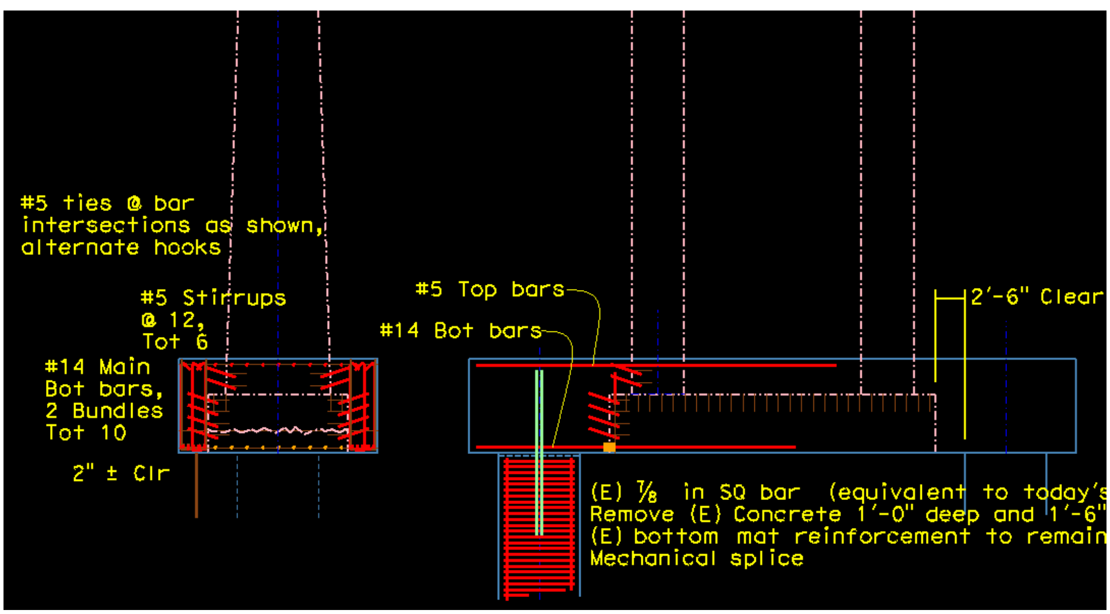
= 370 LF

Total regular Drill and Bond Dowel: $\boxed{\$1,642\ LF}$.



Drill and Bond Dowel – Continued





Drill and Bond Dowel – Continued

☑	511106 - DRILL AND BOND DOWEL	LF	07	1697	$16.00	$16.00	$27152.00	05-03-2017
☑	511106 - DRILL AND BOND DOWEL	LF	07	1697	$32.00	$32.00	$54304.00	05-03-2017
☑	511106 - DRILL AND BOND DOWEL	LF	07	1697	$60.00	$60.00	$101820.00	05-03-2017
☑	511106 - DRILL AND BOND DOWEL	LF	07	1697	$1.00	$1.00	$1697.00	05-03-2017
☑	511106 - DRILL AND BOND DOWEL	LF	04	1400	$50.00	$50.00	$70000.00	06-20-2017
☑	511106 - DRILL AND BOND DOWEL	LF	04	1400	$10.00	$10.00	$14000.00	06-20-2017
☑	511106 - DRILL AND BOND DOWEL	LF	04	1400	$35.00	$35.00	$49000.00	06-20-2017

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	29.70	**33.27**	Avg No. Units	1794
Std Dev. (of Unit Price): ±$	15.28	**16.18**	Rows Selected	39
Weighted Avg.: $	28.95	**32.74**	Rows Returned	39
Minimum Price/Unit: $	1.00	1.00		
Maximum Price/Unit: $	80.00	79.78		

Based on Caltrans Contract Cost Data,
Drill and Bond Dowel has an average adjusted unit price of $35/LF for quantities between 1000 and 2000 LF.

For estimate Say $\boxed{\$40/LF}$

Drill and Bond Dowel (Chemical Adhesive) [LF]

Drill and Bond Dowel (Chemical Adhesive) are required for rebars that are installed at horizontal or vertical. For these bars, chemical adhesive is required to prevent the grout from spilling out.

Tie Girder Bolter: [(4 Rows in vert direction as shown in section) (6 Columns per bay) (15 bays) ((10/12 LF each)(4 quadrant)

= 1200 LF

// back check [(4 Rows in vert direction as shown in section) (100 Columns to install over the 100' long girder per quad – 15 columns for floor beams locations at 1 per location) ((10/12 LF each)(4 quadrant) *= 1134 LF okay*

Approach Span Exterior Girder Bolter: // back check [(4 Rows in vert direction as shown in section) (37 Columns to install over the 36' long girder per quad) ((10/12 LF each)(4 quadrant)

= 494 LF

Pier 2,3,4 Bolster: [(3 Rows in vert direction as shown in section) (17 Rows for 15.167' long per side) ((10/12 LF each)(2 sides)(4 pier supports)

= 340 LF

Floor beam Bolter adjacent to Arch: [(1 Rows in vert direction as shown in section) (17 Rows for 15.167' long per side) ((10/12 LF each)(1 location)(4 location sets)

= 57 LF

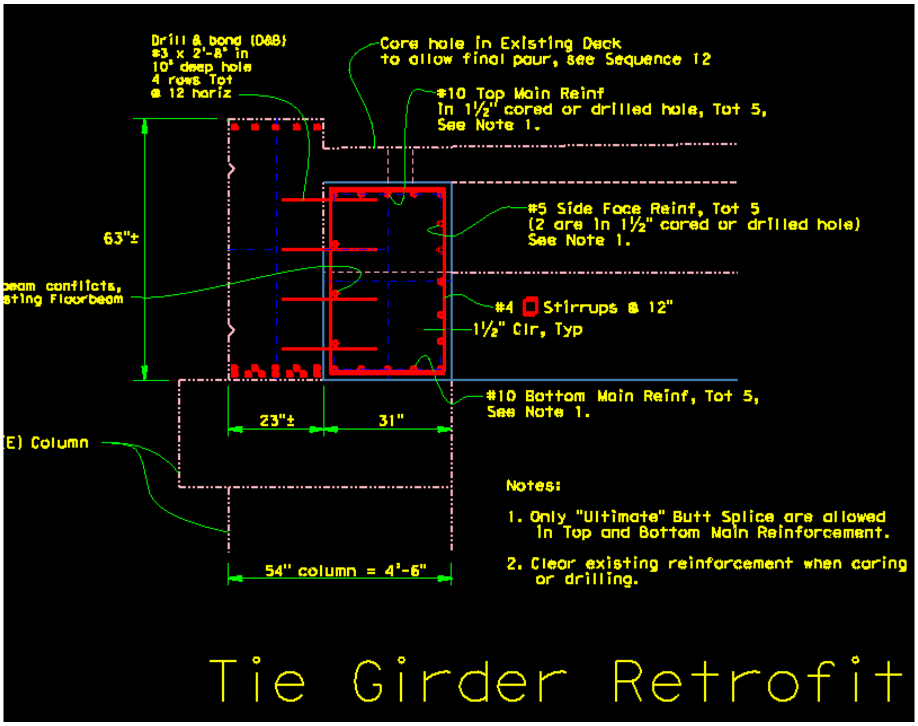
Total Drill and Bond Dowel (Chemical Adhesive), Say: $\boxed{\text{2,092 LF}}$

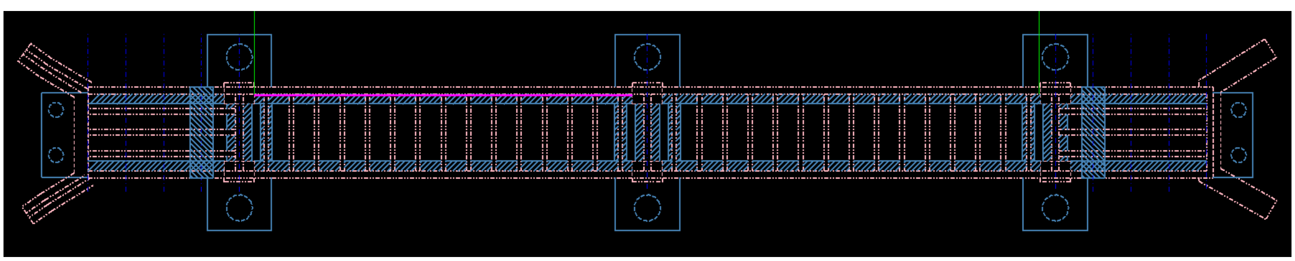
Based on Caltrans Contract Cost Data,

Drill and Bond Dowel (Chemical Adhesive) has an average adjusted unit price of $52/LF.

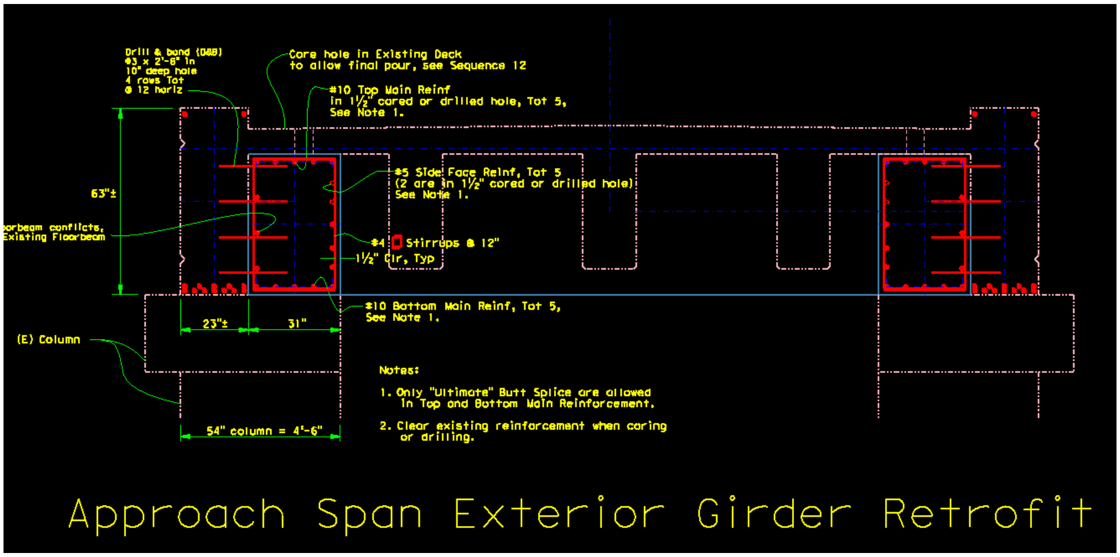
For estimate Say $\boxed{\text{\$55/LF}}$

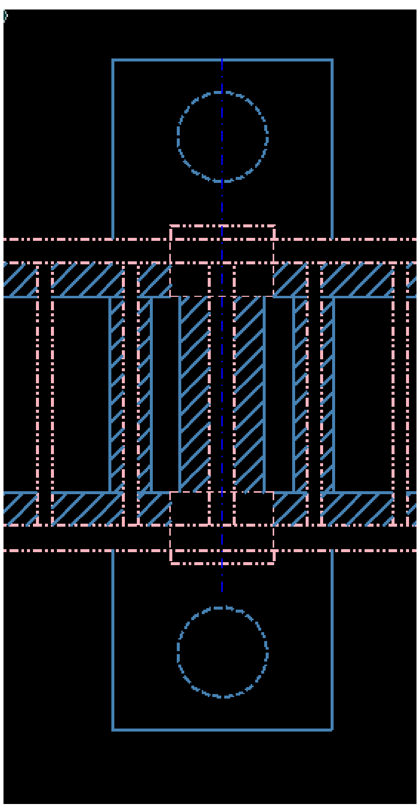
Drill and Bond Dowel (Chemical Adhesive) – Continued

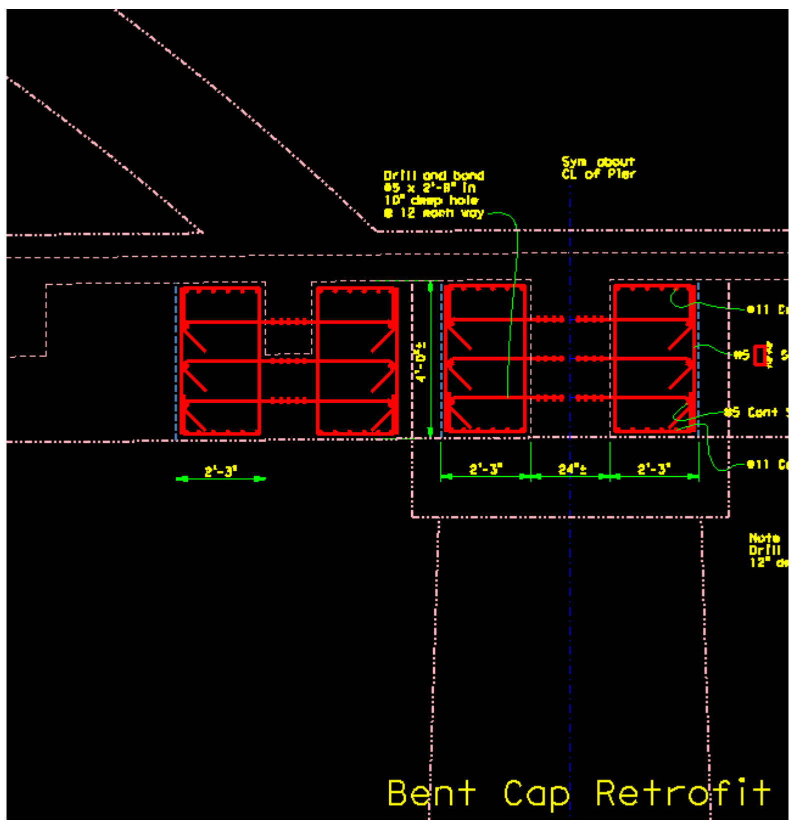




Drill and Bond Dowel (Chemical Adhesive) – Continued







Drill and Bond Dowel (Chemical Adhesive) – Continued

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$25.00	$25.00
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$108.00	$108.00
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$25.00	$25.00
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$75.00	$75.00
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$8.60	$8.60
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$108.00	$108.00
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	07	536	$7.60	$7.60
☑	511111 - DRILL AND BOND DOWEL (CHEMICAL ADHESIVE)(LF)	LF	01	49	$63.00	$63.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	52.52	**52.52**	Avg No. Units	475
Std Dev. (of Unit Price): ±$	39.10	**39.10**	Rows Selected	8
Weighted Avg.: $	51.18	**51.18**	Rows Returned	8
Minimum Price/Unit: $	7.60	7.60		
Maximum Price/Unit: $	108.00	108.00		

Bar Reinforcing Steel (Bridge) [LB]

Density of Steel, 490 lb/CY, is obtained from Bridge Design Aids Ch.11

Abut 1 Bolster $_{\text{CIDH to Abut Stem attachment}}$:

(89 CY) (200 lb / CY) = 18,000 lb

Pier 2 Footing:

(108 CY) (180 lb / CY) = 36,000 lb

Pier 3 Footing:

(108 CY) (180 lb / CY) = 36,000 lb

Pier 4 Footing:

(108 CY) (180 lb / CY) = 36,000 lb

Abut 5 Bolster $_{\text{CIDH to Abut Stem attachment}}$:

(163 CY) (200 lb / CY) = 32,000 lb

The following rebar are approximate for the preliminary phase:

Abut 1 CIDH:

(70 LF) (20 EA #11)(1.56 sq-in) (490 lb/CF) / (12 in)^(2) = 7,500 lb

2 pier = [(60" – 3"x2) /12 ft] [pi] [0.31 sq-in / (12 in)^(2)] [490 lb/CF]

x [70'] / [9'/12] = 1,400 lb

Pier 2 CIDH:

(110 LF) (36 EA #11)(1.56 sq-in) (490 lb/CF) / (12 in)^(2) = 21,000 lb

2 pier = [(84" – 3"x2) /12 ft] [pi] [0.31 sq-in / (12 in)^(2)] [490 lb/CF]

x [110'] / [9'/12] = 3,000 lb

Pier 3 CIDH:

(110 LF) (36 EA #11)(1.56 sq-in) (490 lb/CF) / (12 in)^(2) = 21,000 lb

2 pier = [(84" – 3"x2) /12 ft] [pi] [0.31 sq-in / (12 in)^(2)] [490 lb/CF]

x [110'] / [9'/12] = 3,000 lb

Pier 4 CIDH:

(110 LF) (36 EA #11)(1.56 sq-in) (490 lb/CF) / (12 in)^(2) = 21,000 lb

2 pier = [(84" – 3"x2) /12 ft] [pi] [0.31 sq-in / (12 in)^(2)] [490 lb/CF]

x [110'] / [9'/12] = 3,000 lb

Abut 5 CIDH:

(70 LF) (20 EA #11)(1.56 sq-in) (490 lb/CF) / (12 in)^(2) = 7,500 lb

2 pier = [(70" – 3"x2) /12 ft] [pi] [0.31 sq-in / (12 in)^(2)] [490 lb/CF]

x [70'] / [9'/12] = 1,400 lb

Tie Girder Bolter: (140 CY) (250 lb / CY) = 35,000 lb

Approach Span Exterior Girder Bolter: (55 CY) (250 lb / CY) = 14,000 lb

Pier 2,3,4 Bolster: (30 CY) (250 lb / CY) = 8,000 lb

Floor beam Bolter adjacent to Arch: (6 CY) (250 lb / CY) = 2,000 lb

Approach Span Repair $_{conservative}$: (10 CY) (100 lb / CY) = 1,000 lb

$\sum$ = 308,000 LB

Say $\boxed{\text{310,000 LB}}$

Based on Caltrans Contract Cost Data,

Bar Reinforcing Steel (Bridge) has an average adjusted unit price of $1.3/LB. Given the numerous smaller bars and non standard access assume $\boxed{\text{\$ 1.5 /LB}}$

Inject Crack (Epoxy) [LF]

Inject Crack (Epoxy) estimate includes work done at the 3/4 approach span locations.

Span 1 location total parameter location 85 LF

Span 4 location total parameter location 85 LF

Inject Crack (Epoxy) say: 170 LF

Based on Caltrans Contract Cost Data,
Bar Inject Crack (Epoxy) has an average adjusted unit price of $56/LF.
Say $ 60 /LF

289
289
63
63
63
63
30
30
30
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30
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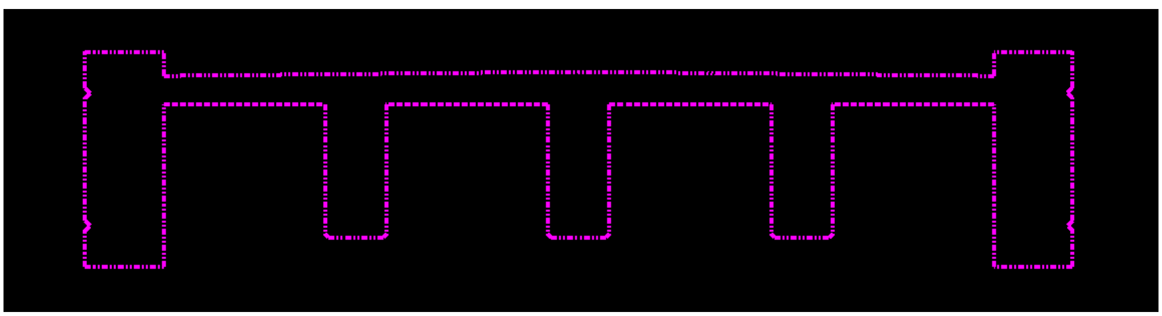
1010
84.16667



	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$55.00	$55.00
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$70.00	$70.00
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$25.00	$25.00
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$60.00	$60.00
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$80.00	$80.00
☑	600003 - INJECT CRACK (EPOXY)	LF	11	180	$46.10	$46.10

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	56.01	**56.01**	Avg No. Units	
Std Dev. (of Unit Price): ±$	17.54	**17.54**	Rows Selected	
Weighted Avg.: $	56.01	**56.01**	Rows Returned	
Minimum Price/Unit: $	25.00	25.00		
Maximum Price/Unit: $	80.00	80.00		

Repair Spalled Surface Area [SQFT]

Work includes removal of unsound concrete, installing of new concrete screws (one at every square foot patch), placing bond coat between existing concrete and patch, and setting concrete patch. Work also includes protecting the existing reinforcing bars and cleaning the reinforcing bars by abrasive blasting. This item covers all spalled areas except the deck.

Girders

Per Alta Vista's assessment report, cracks in the approach spans are considered full depth repairs and should be repaired by removing deteriorated concrete up to six feet north and six feet south of the crack locations, and reconstructing the bridge deck. A section of Alta Vista's report is shown below:

March 31, 2017 *Alta Vista Solutions*

Repair Recommendations for Approach Spans

Assuming no further settlement is anticipated at the approach spans, and no enhanced member capacity is needed, the following repair strategy may be employed. Cracks in the approach spans are considered full depth repairs and should be repaired by removing deteriorated concrete up to six feet north and six feet south of the crack locations, and reconstructing the bridge deck.

For bridge deck, if removal of deteriorated material requires saw-cutting, existing reinforcement should not be damaged. This may be achieved by chipping or hydro blasting, which should employ appropriate equipment that will not damage surrounding concrete or steel. Demolition should result in repair areas that have a step configuration to allow mechanical engagement. Added reinforcement may be required where reinforcement condition appears to be damaged due to settlement, corrosion, or other causes. The Engineer should witness removal operations in order to verify that the extents of damage have been removed, or if further removal is needed. Prior to repairs, surfaces should be cleaned of all substances that would impair bond of repair materials, and an SSD surface condition may be required prior to placement of repair material.

For repair of the girders affected by this cracking, it is recommended that loose material be removed, which may extend 1 inch below the first layer of reinforcement. Areas where cracking is present should be opened to expose sound material. As with the deck, care should be taken to avoid damage to the steel. If the condition of concrete and steel appear deteriorated and extends deeper into girder than is shown from the surface, notify the Engineer to assess the condition and determine an appropriate repair method with additional reinforcement.

Estimated Deck Repair Area:	775 sq.ft.
Girder Estimated Repair Area:	40 sq.ft.
Estimated Reinforcement:	100 ft

Repair Spalled Surface Area [SQFT] is estimated based on the following:

775 SF Deck repair area (is not included for Repair Spalled Surface Area item as it is covered under remove unsound concrete and rapid set concrete patch items.)

40 SF Girder repair area

= 40 SF

Soffits (all four spans)

Category 1 **GOOD**	Generally, no defects identified. No repair required, however it is recommended that visual inspection be performed after any substructure retrofit is complete or as deck repairs are being done to assess whether any additional defects result. At this point, reassessment of defect category must be performed and applicable repairs be performed as needed.
Category 2 **FAIR**	At the locations identified with cracking or rocks pockets/voids, repairs should include removal of unsound concrete, saw-cutting two inches beyond the affected area. Saw-cut for overhead repairs shall be angled to promote mechanical engagement with of repair material with existing. If, during removal of unsound concrete, reinforcement is exposed, follow the repair procedure for Category 3/4. If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010” should be repaired by epoxy injection or other suitable material. Proper surface preparation and bonding agent should be employed based on manufacturer’s recommendations for appropriate patching material.
Category 3 **POOR**	At the locations identified with cracking, exposed reinforcement, or rock pockets/voids, repairs should include removal of unsound concrete, and saw-cutting two inches around the affected area. Saw-cut for overhead repairs shall be angled to promote mechanical engagement with of repair material with existing. In case of exposed rebar, material removal should extend 1 inch beyond the first layer of reinforcement to allow mechanical engagement of repair material.
Category 4 **SEVERE**	After material removal is complete, exposed reinforcement should be cleaned of bond inhibiting agents and concrete should be examined for cracks. If, during removal of concrete it is determined that cross-section loss has occurred, notify the Engineer to determine appropriate repair method. If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010” should be repaired by epoxy injection or other suitable material. Proper surface preparation and bonding agent should be employed based on manufacturer’s recommendations for appropriate patching material.

Estimated Soffit Repair Area:	Category 2:	267 sq.ft.
	Category 3:	380 sq.ft.
	Category 4:	759 sq.ft.

Railings

Railing		Between Abutment 1/Pier 2 (Repair area: lumpsum) Spalled railing posts, exposed reinforcement. *Remove unsound material, clean surface and patch. If majority of section is damaged, individual posts should be replaced in kind.*
		At Arch 1 West (Repair area: lumpsum) Cracking at what appears to be patched area. *Remove unsound material, clean surface and patch.*
		At Arch 3 west (Repair area: lumpsum) Crack and void between railing and arch. *Remove unsound material, clean surface and patch.*
		At Arch 4 East (Repair area: lumpsum) Appears to be an uneven repair area. *Remove unsound or uneven material, clean surface and patch.*

March 31, 2017 *Alta Vista Solutions*

Repair Recommendations for Arches, hangers and railings

Various observations of defects were recorded for the superstructure of the bridge. Individual locations from the superstructure are shown in Table 3.

In general, locations which have exposed reinforcement and spalled or loose material as shown in Figures 9 and 10 need to be repaired, which include removing loose material until sound concrete is encountered, cleaning rebar and concrete substrates, and applying patching material to restore the surface of the member while protecting the rebar from corrosion.

Figure 9 - Exposed rebar at east Abutment 1 railing

Figure 10 - Spalled overhead section of arch

If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010" should be repaired by epoxy injection. Typically, available epoxy products have a range of viscosities available which are able to accommodate repairs to cracks of up to 1/4 inch width.

Table 3 identifies deteriorated areas observed on the superstructure and potential repair strategies that may be used. The Feasibility Study provided recommendations for retrofit including fiber wrap for seismic loading. While fiber wrap is commonly used to increase strength and confinement, the repairs recommended here including patching and fiber wrap are intended to protect the identified element from further deterioration and to restore to as-built conditions.

Estimated Repair Area: 37 sq.ft. plus lumpsum for railing

Estimated railing repair length:

37 FT. (Do not include in total since rail will be replaced)

Arches

Arches		West at Hanger 7 (Repair area: 4 sq.ft.) Exposed reinforcement under arch, cracking and spalling *Remove unsound concrete, clean rebar and patch. Fiber wrap at this location due to proximity to pier.*
		West at Hanger 8 (Repair area: 6 sq.ft.) Exposed reinforcement under arch, cracking, heavy spalling, loose material. *Remove all unsound material, clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*
		West at Hanger 11 (Repair area: 4 sq.ft.) Exposed reinforcement, cracking, some spalling, possible loose material *Remove unsound concrete, clean rebar and patch.*
		East at Hanger 11 (Repair area: 6 sq.ft.) Exposed reinforcement, cracking, spalling, loose material. *Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*

Estimated Arch Rib repair area:

(4 SF + 6 SF + 4 SF + 6 SF) = 20 SF.

Vertical Hangers

Hangers		Hanger 5 West (Repair area: 6 sq.ft.) Exposed reinforcement, heavy spalling, and visible aggregate. *Remove unsound material and bond inhibiting substances. Clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*
		Hanger 11 East (Repair area: 6 sq.ft.) Heavy spalling, cracking, unsound concrete. *Remove all unsound material, clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*

Estimated Vertical Hangers repair area:

(6 SF + 6 SF) = 12 SF.

Portal Bracing

Portal Bracing		Bracing at Hanger 2 (Repair area: 2 sq.ft.) Exposed reinforcement and spalling *Remove unsound concrete, clean rebar and patch. Use fiber wrap or wire reinforcement to secure overhead patch material from falling.*
		Portal Bracing at Hanger 8: (Repair area: 3 sq.ft.) Corner spalling, cracking, exposed reinforcement. *Remove unsound concrete, clean rebar and patch. Use fiber wrap or wire reinforcement to secure overhead patch material from falling.*

Estimated Vertical Hangers repair area:

(2 SF + 3 SF) = 5 SF.

Other Bridge Elements

Other bridge elements may have minor areas of work. The conservatism built-in to the previous estimated bridge elements will capture the minor area of work not accounted for. The bridge substructure generally is in good shape.

Repair Spalled Surface Area								
	Approach Spans							
		775	SF	Deck				
		40	SF	Girder				
		40	**SF**	**Approach Span**				
	Approach Spans							
		775	0.8	620	CF			
		40	0.25	10	CF			
				630	**CF**	**Approach Span**		
	Soffits (all four spans)							
		267	SF	Fair				
		380	SF	Poor				
		759	SF	Server				
		1406	**SF**	**Soffits**				
	Soffits (all four spans)							
		267	0.1667	45	CF			
		380	0.25	95	CF			
		759	0.3333	253	CF			
				392	**CF**	**Soffits**		
	Arch Ribs							
		4	SF	Deck				
		6	SF	Girder				
		4	SF	Girder				
		6	SF	Girder				
		20	**SF**	**Arch**				
	Arch Ribs							
		4	0.3333	1	CF			
		6	0.3333	2	CF			
		4	0.3333	1	CF			
		6	0.3333	2	CF			
				7	**CF**	**Arch**		
	Verticals							
		6	SF					
		6	SF					
		12	**SF**	**Verticals**				
	Verticals							
		6	0.3333	2	CF			
		6	0.3333	2	CF			
				4	**CF**	**Verticals**		
	Portal							
		2	SF					
		3	SF					
		5	**SF**	**Verticals**				
	Portal							
		2	0.25	1	CF			
		3	0.25	1	CF			
				1.3	**CF**	**Verticals**		
	Total	**1483**	**SF**	**Repair Spalled Surface Area**				

Total estimated Repair Spalled Surface Area is 1,483 SF. The covers repairs up to 4" deep. The deck area is paid separately under the Remove Unsound Concrete and Rapid Setting Concrete (Patch) items. Given the possibility of repairs deeper than 4" or larger areas after the removal of unsound concrete increase area by 25%. Assume 1,854 SF

Based on Caltrans bid history, below are the average adjusted average Repair Spalled Surface Area costs for three different Bid Item numbers.

For Caltrans Bid Item #600013, the unit cost is around $440/SF.

For Caltrans Bid Item #150312, the unit cost is around $150/SF.

For Caltrans Bid Item #515028, the unit cost is around $130/SF.

Accounting for project size, the type of repair (on existing Bridge) and the level of effort, the estimated unit cost is:

Unit Cost = $\boxed{\$440/SF}$

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price	Total	Bid Open Date	Contract No.	Bid	M	TRO
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	25	$110.00	$110.00	$2750.00	04-25-2017	04-4J5604	1	M	
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	25	$375.00	$375.00	$9375.00	04-25-2017	04-4J5604	2	M	
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	25	$660.00	$660.00	$16500.00	04-25-2017	04-4J5604	3	M	
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	27	$640.00	$640.00	$17280.00	05-02-2017	04-4J5804	1	M	
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$600.00	$600.00	$13200.00	05-09-2017	04-4J6004	1	M	TRO
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$350.00	$350.00	$7700.00	05-09-2017	04-4J6004	2	M	TRO
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$525.00	$525.00	$11550.00	05-09-2017	04-4J6004	3	M	TRO
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$257.00	$257.00	$5654.00	05-09-2017	04-4J6004	4	M	TRO
☑	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$400.00	$400.00	$8800.00	05-09-2017	04-4J6004	5	M	TRO
☐	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	22	$65.00	$65.00	$1430.00	05-09-2017	04-4J6004	6	M	TRO
☐	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	37	$900.00	$900.00	$33300.00	05-10-2017	04-4J7004	1	M	
☐	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	37	$1000.00	$1000.00	$37000.00	05-10-2017	04-4J7004	2	M	
☐	600013 - REPAIR SPALLED SURFACE AREA	SQFT	04	37	$5100.00	$5100.00	$188700.00	05-10-2017	04-4J7004	3	M	

uncheck all | check all cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	435.22	**435.22**	Avg No. Units	23
Std Dev. (of Unit Price): ±$	175.54	**175.54**	Rows Selected	9
Weighted Avg.: $	437.77	**437.77**	Rows Returned	13

☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	600	$200.00	$222.30	$120000.00	05-04-2016	07-302604	7	M	TRO
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$115.30	$115.30	$57534.70	02-22-2017	07-3W1804	1	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$70.00	$70.00	$34930.00	02-22-2017	07-3W1804	2	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$115.00	$115.00	$57385.00	02-22-2017	07-3W1804	3	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$80.00	$80.00	$39920.00	02-22-2017	07-3W1804	4	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$140.00	$140.00	$69860.00	02-22-2017	07-3W1804	5	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$197.24	$197.24	$98422.76	02-22-2017	07-3W1804	6	M	
☑	150312 - REPAIR SPALLED SURFACE AREA	SQFT	07	499	$135.00	$135.00	$67365.00	02-22-2017	07-3W1804	7	M	

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SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	119.62	**144.59**	Avg No. Units	NaN
Std Dev. (of Unit Price): ±$	80.41	**95.18**	Rows Selected	145
Weighted Avg.: $	0.00	**125.51**	Rows Returned	145
Minimum Price/Unit: $	0.00	0.00		
Maximum Price/Unit: $	400.00	507.48		

☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	11	330	$110.00	$209.24	$36300.00	07-12-2012	11-270804	3	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	11	330	$64.00	$121.74	$21120.00	07-12-2012	11-270804	4	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	11	330	$60.00	$114.13	$19800.00	07-12-2012	11-270804	5	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$52.00	$98.92	$76024.00	08-23-2012	07-2X8404	1	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$15.00	$28.53	$21930.00	08-23-2012	07-2X8404	2	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$10.00	$19.02	$14620.00	08-23-2012	07-2X8404	3	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$85.00	$161.69	$124270.00	08-23-2012	07-2X8404	4	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$39.60	$75.33	$57895.20	08-23-2012	07-2X8404	5	M	
☑	515028 - REPAIR SPALLED SURFACE AREA	SQFT	07	1462	$47.00	$89.40	$68714.00	08-23-2012	07-2X8404	6	M	

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cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	79.44	**133.43**	Avg No. Units	1348
Std Dev. (of Unit Price): ±$	52.85	**80.50**	Rows Selected	63
Weighted Avg.: $	70.16	**121.47**	Rows Returned	63
Minimum Price/Unit: $	10.00	19.02		
Maximum Price/Unit: $	267.00	382.30		

Previous Estimate

For comparison only, in 2007 the estimated Stevenson Bridge unit price for Repair Spalled Surface Area is $200/SF.

TRC Imbsen JOB NO. 0501334-0202

_X_ PLANNING ESTIMATE _____ BRIDGE GENERAL PLAN ESTIMATE ____ 60% ESTIMATE

Bridge: **Stevenson Bridge Road Bridge (Opt. 1)** Br. No.: 23C-0092
Type: Concrete Tied Arch District : 3 County: Sol/Yol Route: Local PM: N/A
No. Spans: (4) Four Width (ft) 24.17 Length (ft) 296 Area (ft2) 7154
Quantities - CJP 07/21/06 Pricing - MRP 08/03/06 Rev KTN 12/20/06

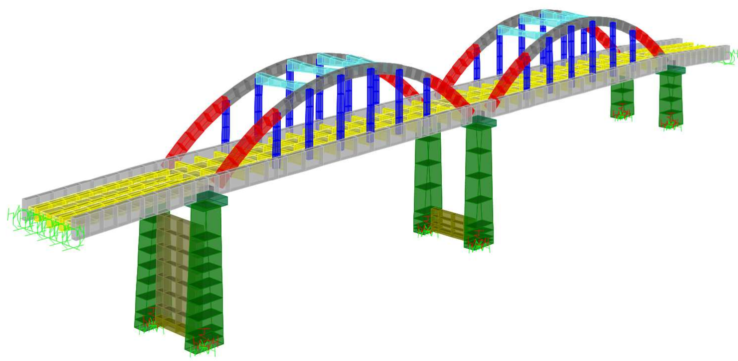
	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	Structure Excavation (Bridge)	CY	1000	$150.00	$150,000.00
2	Structure Backfill, Bridge	CY	500	$120.00	$60,000.00
3	Refinish Bridge Railing	LF	647	$150.00	$97,050.00
4	Bridge Removal (Portion), Curtain Walls	SQFT	800	$20.00	$16,000.00
5	Bridge Removal (Portion), Hanger Column	EA	1	$15,000.00	$15,000.00
6	Bridge Removal (Portion), Approach Slab	SQFT	410	$25.00	$10,250.00
7	Remove Unsound Concrete	EA	100	$100.00	$10,000.00
8	Repair Spalled Surface Area	SQFT	100	$200.00	$20,000.00
9	Structural Concrete (Bridge)	CY	540	$1,300.00	$702,000.00
10	Bar Reinforcing Steel (Bridge)	LB	108000	$2.00	$216,000.00
11	Fiber-Wrap	SQFT	6492	$50.00	$324,600.00
12	Reconstruct Drains	EA	60	$200.00	$12,000.00
13	Clean Bridge Deck	SQFT	6068	$5.00	$30,340.00
14	Furnish Polyester Concrete Overlay (1")	CY	19	$3,000.00	$57,000.00
15	Place Polyester Concrete Overlay	SQFT	6068	$10.00	$60,680.00
16	60" Cast-In-Drilled-Hole Concrete Piling	LF	200	$900.00	$180,000.00
17	84" Cast-In-Drilled-Hole Concrete Piling	LF	570	$2,800.00	$1,596,000.00
18					

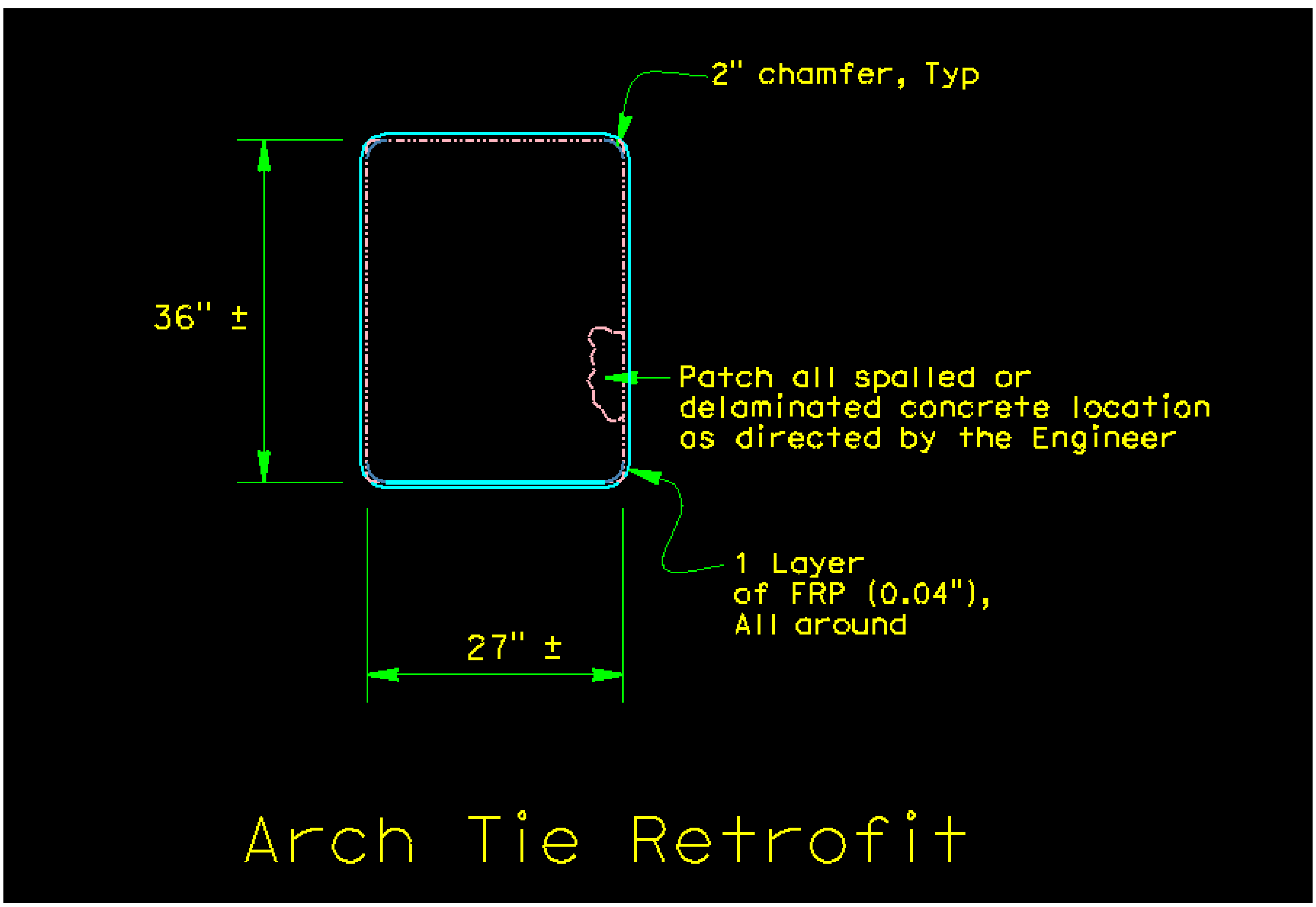
Fiber Wrap [SQFT]

Arch Rib – FRP Area Estimation:

(25 ft Arch Rib length at CL of arch from Spring line to first column EA) [(27”/12’ width of Arch) (2 sides per section) + (36”/12’ depth of Arch)) (2 per section)] (4 sections per span) (2 Spans) = 1,850 SF

(1,850 SF) (1 layers) = 1,850 SF





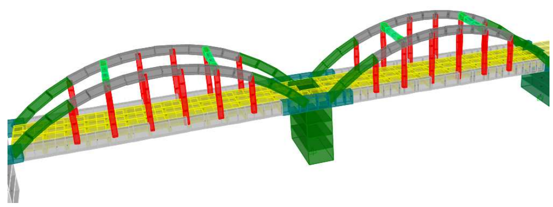
Vertical Hanger – FRP Area Estimation:

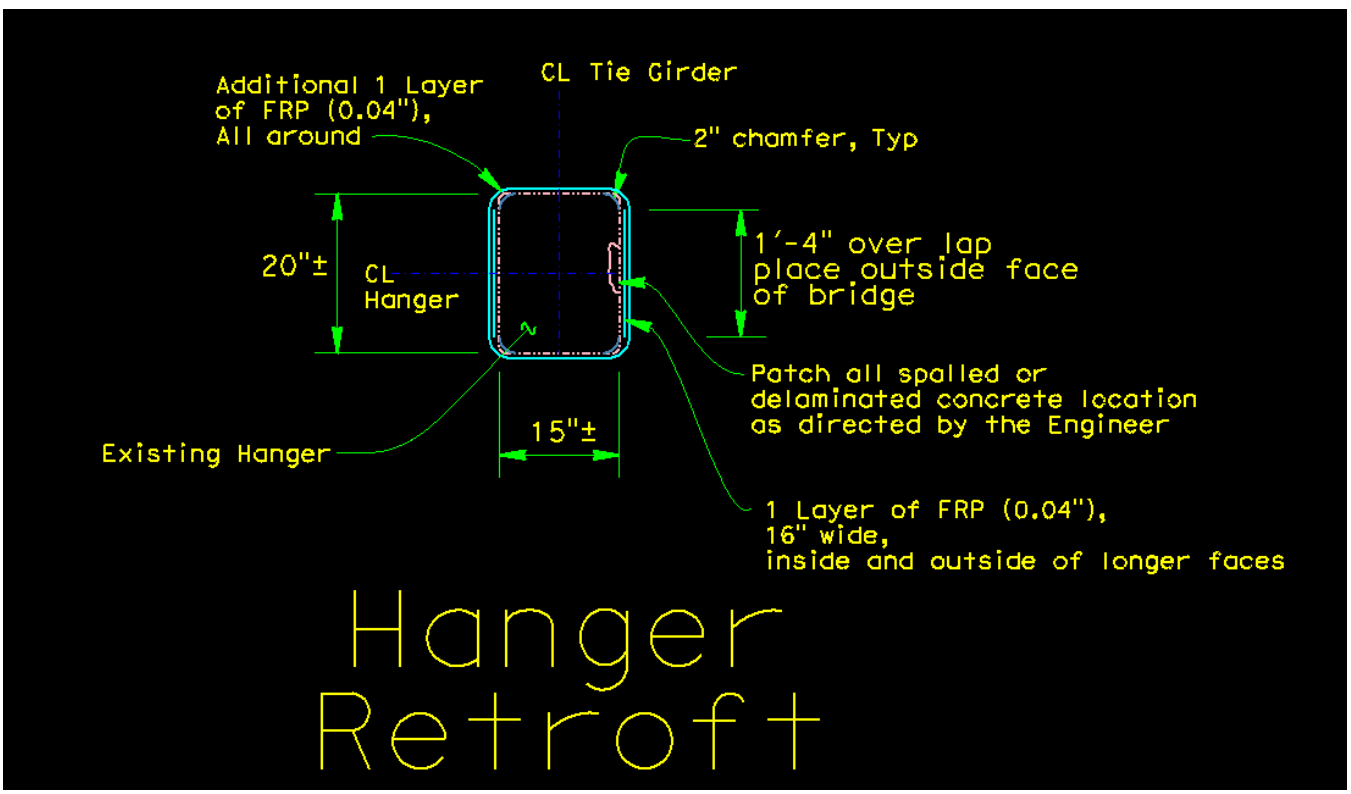
Average vertical hanger length = (14.3' + 18.7' + 20.8' + 20.8' + 18.7' 14.3') / 6 = 18'

(18 ft average Vertical Hanger length EA) [(20"/12' width of Hanger) (2 sides) + (15"/12' width of Hanger) (2 sides)] (6 Hanger per arch) (2 arches per span) (2 Spans) = 2,520 SF for confinement.

(18 ft average Vertical Hanger length EA + 4 feet for development) [(20"/12' width of Hanger) (2 sides)] (6 Hanger per arch) (2 arches per span) (2 Spans) = 1,760 SF for Strengthening.

2,520 SF + 1,760 SF= 4,280 SF





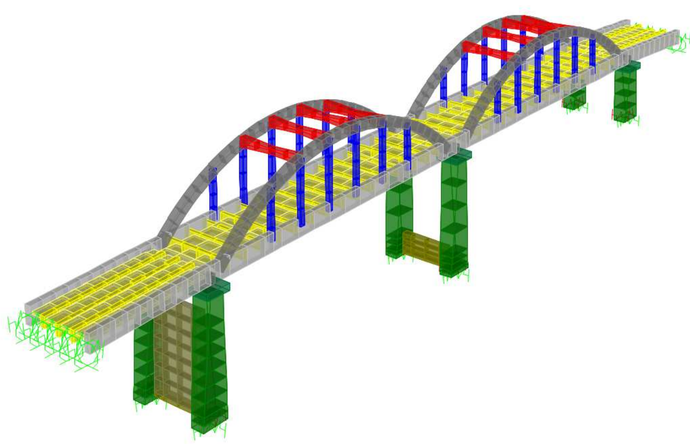
Portal Bracing – FRP Area Estimation:

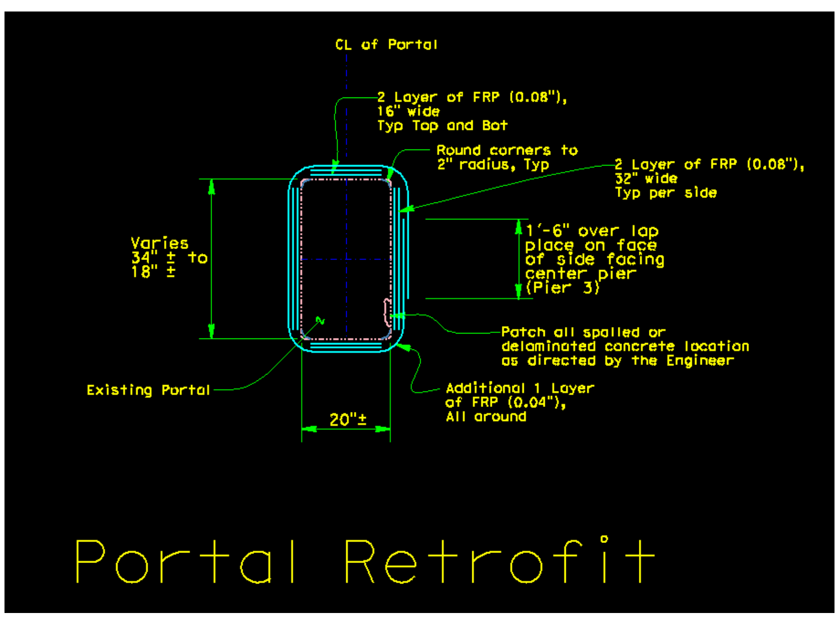
Top and Bottom Layers: (22.5 ft Portal Bracing length + 22.5 ft Portal Bracing length + 3 ft overlap on the arch + 3 ft overlap on the arch + 2 ft overlap on the top side + 2 ft overlap on the bottom side) (20”/12’ width of Portal in plan view) (2 layers) = 184 SF for strength for one Portal.

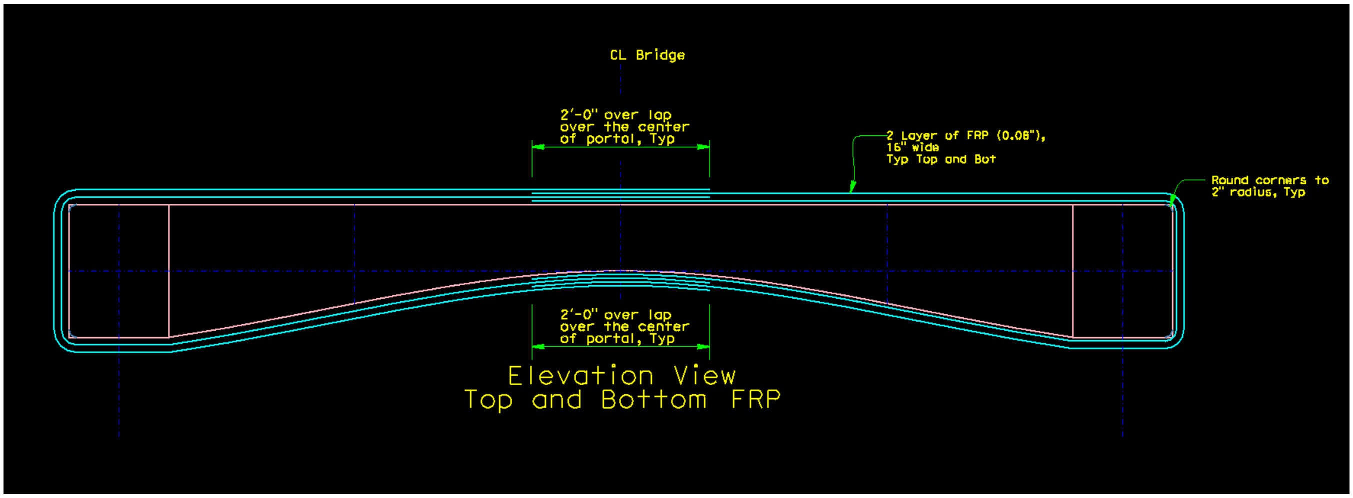
Side Layers: (22.5 ft Portal Bracing length + 22.5 ft Portal Bracing length + 20/12 ft overlap on the arch + 20/12 ft overlap on the arch + 2 ft overlap on the one side + 2 ft overlap on the other side) (26”/12’ average height of Portal in elevation view) (2 layers) = 227 SF for strength for one Portal.

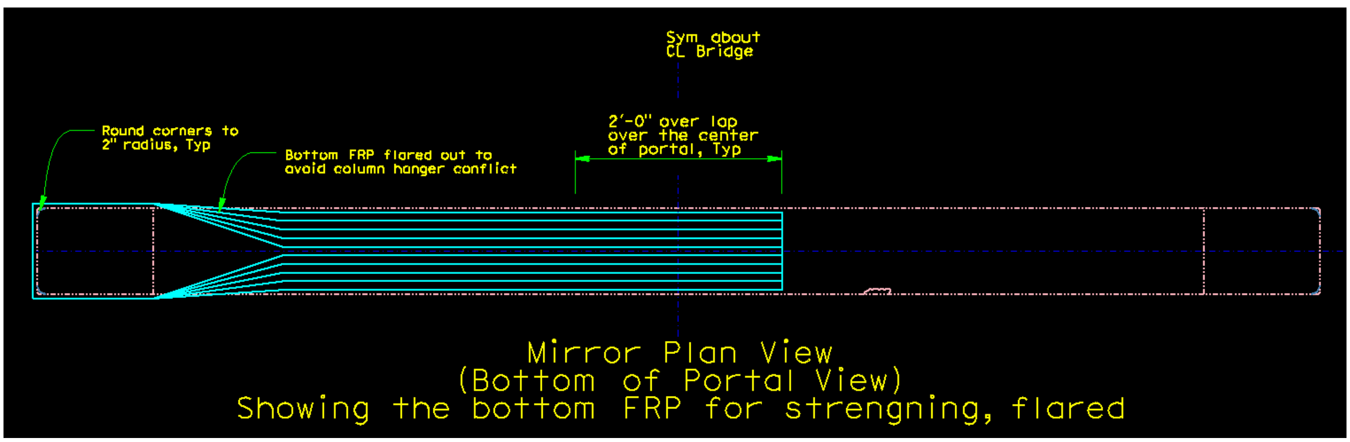
Confinement Layer: (20.3333 ft length to be confined) [(3' depth of Arch) (2 sides) + (20"/12' width of portal)) (2 sides)] = 190 SF

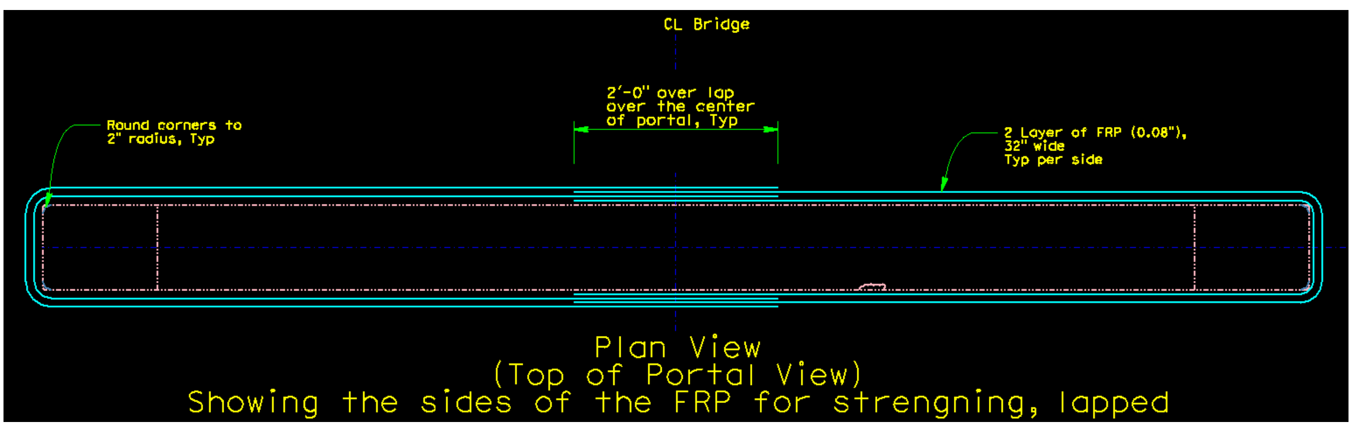
184 SF + 227 SF +190 SF = 600 SF per Portal x 4 portals = 2,400 SF





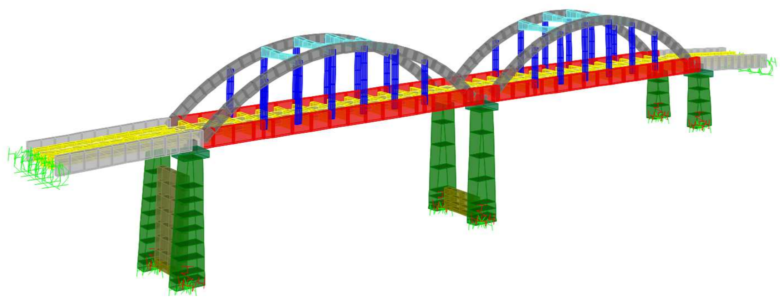

CL Bridge
2'-0" over lap over the center of portal, Typ
2 Layer of FRP (0.08"), 16" wide Typ Top and Bot
Round corners to 2" radius, Typ
2'-0" over lap over the center of portal, Typ
Elevation View
Top and Bottom FRP


Sym about CL Bridge
Round corners to 2" radius, Typ
Bottom FRP flared out to avoid column hanger conflict
2'-0" over lap over the center of portal, Typ
Mirror Plan View
(Bottom of Portal View)
Showing the bottom FRP for strengning, flared


CL Bridge
Round corners to 2" radius, Typ
2'-0" over lap over the center of portal, Typ
2 Layer of FRP (0.08"), 32" wide Typ per side
Plan View
(Top of Portal View)
Showing the sides of the FRP for strengning, lapped

Tie Girder – FRP Area Estimation:

Tie Girder are proposed to be bolstered with concrete sections. FRP is not proposed.



Total Fiber Wrap Area Estimation:

1,850 SF Arch Ribs + 4,280 SF Vertical Hangers + 2,400 SF Portal = say **8,530 SF**

Estimated price = 2014 unit price is $ 26.50/SF, based on Fyfe's presentation workshop info below. Accounting for 3% inflation per year. FV = PV (1 + r)^n = (26.50)(1.03)^4 = $30/SF. Accounting for Prime Contractor's mark-up to provide overhead and access to Fiber Wrap Sub say **$60 /SF**

While the cost varies significantly based on the location of the work and how much work is required, Fyfe provided the following common cost of wrap:

$25/SF per carbon Layer

Add $1-2/SF for UV protection

Add $5-15/SF for fire rating protection (for information only. Not required for this project)

Based on Caltrans bid history, there have been few FRP projects. The FRP bid item names also are different. The average adjusted prices are listed below. The FRP unit costs obtained from Caltrans bid history are for information only.

GLASS FIBER REINFORCED POLYMER REPAIR (EPOXY INJECTION)	Price $77/SF	Qty 50 SF
WET LAY-UP GLASS FIBER REINFORCED POLYMER COMPOSITE	Price $60/SF	Qty 1650 SF
FIBER REINFORCED POLYMER STRIP	Price $30/SF	Qty 2950 SF
PREPARE FIBER REINFORCED POLYMER DECK SURFACE	Price $2/SF	Qty 8050 SF
FURNISH FIBER REINFORCED POLYMER DECK PANEL 5" THICK	Price $116/SF	Qty 8600 SF
FIBER REINFORCED PLASTIC (FRP) DECKING	Price $60/SF	Qty 60 SF
610 FIBER REINFORCED PLASTIC (FRP) DECKING	Price $60/SF	Qty 60 SF

Previous Estimate

For comparison only, in 2007 the estimated Stevenson Bridge unit price for Fiber-Wrap was $50/SF, with an estimated area of repair of 6,492 SF. The total estimated cost was $325k.

Remove Unsound Concrete [CF] and Rapid Set Concrete Patch [CF]

Remove Unsound Concrete and Rapid Set Concrete Patch bid items are for the deck area only (Removal of unsound concrete and patches for non-deck spalls are covered in the Repair Spalled Surface Area bid item). Alta Vista estimated the area of deck repairs at 775 SF. Increase the area by 25% to account for unforeseen unsound concrete area. Also to be conservative assume the depth of the repair is the entire deck thickness.

Remove Unsound Concrete [CF] and Rapid Set Concrete Patch [CF] is estimated as following.

(1.25)(775 SF Deck repair area) (0.8 FT thick deck)

= 775 CF

As shown in the previous calculations, the estimated quantity is = 775 CF for both bid items

Use = $\boxed{\text{775 CF}}$

Based on Caltrans bid history, the average adjusted Remove Unsound Concrete Area is around $113/CF.

Accounting for project size, the type of repair (on existing Bridge) and the level of effort, the estimated unit cost is:

Estimated price = $\boxed{\text{\$120/CF}}$

600033 - REMOVE UNSOUND CONCRETE	CF	06	137	$100.00	$100.00	$13700.00	03-29-2017	06-0U1504	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	07	157	$50.00	$50.00	$7850.00	05-03-2017	07-3W2004	4	M	
600033 - REMOVE UNSOUND CONCRETE	CF	07	157	$171.00	$171.00	$26847.00	05-03-2017	07-3W2004	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	07	157	$120.00	$120.00	$18840.00	05-03-2017	07-3W2004	1	M	
600033 - REMOVE UNSOUND CONCRETE	CF	07	157	$75.00	$75.00	$11775.00	05-03-2017	07-3W2004	2	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	165	$60.00	$60.00	$9900.00	02-01-2017	05-1H0804	4	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	165	$94.99	$94.99	$15673.35	02-01-2017	05-1H0804	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	165	$53.00	$53.00	$8745.00	02-01-2017	05-1H0804	1	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	165	$30.00	$30.00	$4950.00	02-01-2017	05-1H0804	2	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	210	$115.00	$115.00	$24150.00	01-24-2017	05-1H0704	4	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	210	$64.00	$64.00	$13440.00	01-24-2017	05-1H0704	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	210	$55.00	$55.00	$11550.00	01-24-2017	05-1H0704	1	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	210	$80.00	$80.00	$16800.00	01-24-2017	05-1H0704	5	M	
600033 - REMOVE UNSOUND CONCRETE	CF	05	210	$41.00	$41.00	$8610.00	01-24-2017	05-1H0704	2	M	
600033 - REMOVE UNSOUND CONCRETE	CF	02	269	$197.11	$197.11	$53022.59	03-14-2017	02-1H8804	1	M	
600033 - REMOVE UNSOUND CONCRETE	CF	02	269	$82.00	$82.00	$22058.00	03-14-2017	02-1H8804	2	M	
600033 - REMOVE UNSOUND CONCRETE	CF	02	269	$95.00	$95.00	$25555.00	03-14-2017	02-1H8804	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	02	269	$27.00	$27.00	$7263.00	03-14-2017	02-1H8804	4	M	
600033 - REMOVE UNSOUND CONCRETE	CF	02	269	$100.00	$100.00	$26900.00	03-14-2017	02-1H8804	5	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$75.00	$75.00	$24075.00	03-07-2017	08-1G3904	4	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$40.00	$40.00	$12840.00	03-07-2017	08-1G3904	6	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$25.00	$25.00	$8025.00	03-07-2017	08-1G3904	2	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$25.00	$25.00	$8025.00	03-07-2017	08-1G3904	5	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$64.50	$64.50	$20704.50	03-07-2017	08-1G3904	7	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$216.73	$216.73	$69570.33	03-07-2017	08-1G3904	3	M	
600033 - REMOVE UNSOUND CONCRETE	CF	08	321	$88.00	$88.00	$28248.00	03-07-2017	08-1G3904	1	M	
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$65.00	$65.00	$142740.00	05-09-2017	04-4J6004	4	M	TRO
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$42.00	$42.00	$92232.00	05-09-2017	04-4J6004	3	M	TRO
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$20.00	$20.00	$43920.00	05-09-2017	04-4J6004	5	M	TRO
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$30.00	$30.00	$65880.00	05-09-2017	04-4J6004	2	M	TRO
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$35.00	$35.00	$76860.00	05-09-2017	04-4J6004	1	M	TRO
600033 - REMOVE UNSOUND CONCRETE	CF	04	2196	$55.00	$55.00	$120780.00	05-09-2017	04-4J6004	6	M	TRO

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cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	113.82	**113.82**	Avg No. Units	266
Std Dev. (of Unit Price): ±$	101.22	**101.22**	Rows Selected	85
Weighted Avg.: $	63.69	**63.69**	Rows Returned	85
Minimum Price/Unit: $	12.00	12.00		
Maximum Price/Unit: $	650.00	650.00		

Based on Caltrans bid history, the average adjusted Rapid Setting Concrete (Patch) is around \$78/CF.

The estimated unit cost is:

Estimated price = $\boxed{\$80/CF}$

☑	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	07	943	$25.00	$25.10	$23575.00	10-13-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	07	943	$50.00	$50.21	$47150.00	10-13-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	07	943	$35.00	$35.15	$33005.00	10-13-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	07	943	$47.25	$47.45	$44556.75	10-13-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	07	943	$225.43	$226.36	$212580.49	10-13-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$30.00	$30.12	$33120.00	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$93.50	$93.89	$103224.00	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$20.00	$20.08	$22080.00	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$55.00	$55.23	$60720.00	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$151.24	$151.87	$166968.96	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	03	1104	$150.00	$150.62	$165600.00	11-03-2016
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	04	312	$25.00	$25.00	$7800.00	01-11-2017
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	04	312	$25.00	$25.00	$7800.00	01-11-2017
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	04	312	$26.00	$26.00	$8112.00	01-11-2017
☑	150310 - RAPID SETTING CONCRETE (PATCH)	CF	04	312	$175.00	$175.00	$54600.00	01-11-2017

ncheck all \| check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	71.48	**78.66**	Avg No. Units	649
Std Dev. (of Unit Price): ±$	71.16	**71.55**	Rows Selected	85
Weighted Avg.: $	66.36	**73.23**	Rows Returned	85
Minimum Price/Unit: $	10.00	9.97		
Maximum Price/Unit: $	465.00	463.72		

Previous Estimate

For comparison only, in 2007 the Stevenson Bridge unit price for Repair Spalled Surface was $100/EA. The estimated number of spalls was 100, which totals to $10k.

TRC Imbsen JOB NO. 0501334-0202

_X_ PLANNING ESTIMATE ____ BRIDGE GENERAL PLAN ESTIMATE ___ 60% ESTIMATE

Bridge: **Stevenson Bridge Road Bridge (Opt. 1)** Br. No.: 23C-0092
Type: Concrete Tied Arch District : 3 County: Sol/Yol Route: Local PM: N/A
No. Spans: (4) Four Width (ft) 24.17 Length (ft) 296 Area (ft2) 7154
Quantities - CJP 07/21/06 Pricing - MRP 08/03/06 Rev KTN 12/20/06

	CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	Structure Excavation (Bridge)	CY	1000	$150.00	$150,000.00
2	Structure Backfill, Bridge	CY	500	$120.00	$60,000.00
3	Refinish Bridge Railing	LF	647	$150.00	$97,050.00
4	Bridge Removal (Portion), Curtain Walls	SQFT	800	$20.00	$16,000.00
5	Bridge Removal (Portion), Hanger Column	EA	1	$15,000.00	$15,000.00
6	Bridge Removal (Portion), Approach Slab	SQFT	410	$25.00	$10,250.00
7	Remove Unsound Concrete	EA	100	$100.00	$10,000.00
8	Repair Spalled Surface Area	SQFT	100	$200.00	$20,000.00
9	Structural Concrete (Bridge)	CY	540	$1,300.00	$702,000.00
10	Bar Reinforcing Steel (Bridge)	LB	108000	$2.00	$216,000.00
11	Fiber-Wrap	SQFT	6492	$50.00	$324,600.00
12	Reconstruct Drains	EA	60	$200.00	$12,000.00
13	Clean Bridge Deck	SQFT	6068	$5.00	$30,340.00
14	Furnish Polyester Concrete Overlay (1")	CY	19	$3,000.00	$57,000.00
15	Place Polyester Concrete Overlay	SQFT	6068	$10.00	$60,680.00
16	60" Cast-In-Drilled-Hole Concrete Piling	LF	200	$900.00	$180,000.00
17	84" Cast-In-Drilled-Hole Concrete Piling	LF	570	$2,800.00	$1,596,000.00
18					

Prepare Concrete Bridge Deck Surface [SQFT]

Bridge length = 40' + 108' + 108' +40' = 296'

Bridge Area = (296' Total Bridge Length from Abut1 to Abut5) (20’ deck width) = 5,920 SF

Say $\boxed{\text{5,920 SF}}$

Estimated price = $\boxed{\text{\$4/SF}}$

Based on Caltrans bid history, the average adjusted Prepare Concrete Bridge Deck Surface is $1/SF with a maximum unit price of $9/SF. The previous name of this item was Clean Bridge Deck which has much more cost data available. Accounting for remote location and inflation associated with the old clean bridge deck item, use $4/SF for Stevenson Bridge.

☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.15	$0.15
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.13	$0.13
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.15	$0.15
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.18	$0.18
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.15	$0.15
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	670514	$0.16	$0.16
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.20	$0.20
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.17	$0.17
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.16	$0.16
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.55	$0.55
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.31	$0.31
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.20	$0.20
☑	600037 - PREPARE CONCRETE BRIDGE DECK SURFACE	SQFT	11	537833	$0.12	$0.12

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	0.39	**0.39**	Avg No. Units	197013
Std Dev. (of Unit Price): ±$	0.84	**0.84**	Rows Selected	181
Weighted Avg.: $	0.20	**0.20**	Rows Returned	181
Minimum Price/Unit: $	0.07	0.07		
Maximum Price/Unit: $	9.10	9.10		

Old item calculated: Clean Bridge Deck [SQFT]

Based on Caltrans bid history, the average adjusted Clean Bridge Deck is $1/SF with a maximum unit price of $12/SF.

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price	Total	Bid Open Date	Contract No.	Bid	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	07	1575	$2.00	$6.11	$3150.00	04-17-1997	07-164114	1	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	11	1990	$1.37	$3.53	$2738.00	05-29-2003	11-241224	1	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	10	2840	$0.74	$0.89	$2112.00	03-13-2007	10-3A2304	1	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	10	2840	$0.56	$0.67	$1584.00	03-13-2007	10-3A2304	2	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	10	2840	$1.39	$1.66	$3960.00	03-13-2007	10-3A2304	3	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$1.50	$1.99	$2475.00	07-10-2008	12-0E0704	1	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$2.00	$2.66	$3300.00	07-10-2008	12-0E0704	2	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$2.50	$3.32	$4125.00	07-10-2008	12-0E0704	3	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$3.00	$3.99	$4950.00	07-10-2008	12-0E0704	4	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$1.50	$1.99	$2475.00	07-10-2008	12-0E0704	5	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$3.00	$3.99	$4950.00	07-10-2008	12-0E0704	6	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$5.00	$6.65	$8250.00	07-10-2008	12-0E0704	7	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$3.50	$4.65	$5775.00	07-10-2008	12-0E0704	8	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$9.50	$12.63	$15675.00	07-10-2008	12-0E0704	9	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	12	1650	$4.50	$5.98	$7425.00	07-10-2008	12-0E0704	10	M	TRO
☑	153235 - CLEAN BRIDGE DECK	SQFT	01	1580	$1.15	$1.96	$1817.00	05-04-2011	01-493604	1	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	01	1580	$0.85	$1.45	$1343.00	05-04-2011	01-493604	2	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	01	1580	$0.20	$0.34	$316.00	05-04-2011	01-493604	3	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	01	1580	$3.00	$5.12	$4740.00	05-04-2011	01-493604	4	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$2.00	$3.57	$5152.00	12-01-2011	08-0E4104	1	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$2.00	$3.57	$5152.00	12-01-2011	08-0E4104	2	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$0.80	$1.43	$2060.80	12-01-2011	08-0E4104	3	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$0.75	$1.34	$1932.00	12-01-2011	08-0E4104	4	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$2.50	$4.46	$6440.00	12-01-2011	08-0E4104	5	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$2.35	$4.20	$6053.60	12-01-2011	08-0E4104	6	M	
☑	153235 - CLEAN BRIDGE DECK	SQFT	08	2576	$3.50	$6.25	$9016.00	12-01-2011	08-0E4104	7	M	

uncheck all | check all — cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	2.35	**3.63**	Avg No. Units	2036
Std Dev. (of Unit Price): ±$	1.85	**2.54**	Rows Selected	26
Weighted Avg.: $	2.20	**3.45**	Rows Returned	26
Minimum Price/Unit: $	0.20	0.34		
Maximum Price/Unit: $	9.50	12.63		

Previous Estimate

For comparison only, in 2007 the Stevenson Bridge unit price for Clean Bridge Deck was estimated at $5/SF, the estimated area of repair was 6,068 SF which totals to $30k.

Treat Bridge Deck [SQFT]

Bridge length = 40' + 108' + 108' +40' = 296'

Bridge Area = (296' Total Bridge Length from Abut1 to Abut5) (20’ deck width) = 5,920 SF

Estimated price = $\boxed{\$1/SQFT}$

Based on Caltrans bid history, the average adjusted Treat Bridge Deck is $0.9/SF with a maximum unit price of $2/SF. Accounting for remote location and inflation, use $1/SF for Stevenson Bridge.

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price	Total
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$0.50	$0.50	$3360.00
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$0.50	$0.50	$3360.00
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$0.65	$0.65	$4368.00
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$0.75	$0.75	$5040.00
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$2.00	$2.00	$13440.00
☑	600045 - TREAT BRIDGE DECK	SQFT	04	6720	$1.00	$1.00	$6720.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	0.90	**0.90**	Avg No. Units	6720
Std Dev. (of Unit Price): ±$	0.52	**0.52**	Rows Selected	6
Weighted Avg.: $	0.90	**0.90**	Rows Returned	6
Minimum Price/Unit: $	0.50	0.50		
Maximum Price/Unit: $	2.00	2.00		

Previous item estimated Crack Treatment (Methacrylate) [SQYD]

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price	Total	Bid Open Date	Contract No.	Bid	M	TRO
☑	040169 - METHACRYLATE SEAL CONCRETE SURFACES	SQYD	03	636	$25.08	$35.02	$15960.00	04-12-2006	03-0E9004	1		
☑	040169 - METHACRYLATE SEAL CONCRETE SURFACES	SQYD	03	636	$95.32	$133.07	$60648.00	04-12-2006	03-0E9004	2		
☑	040169 - METHACRYLATE SEAL CONCRETE SURFACES	SQYD	03	636	$22.58	$31.52	$14364.00	04-12-2006	03-0E9004	3		
☑	040169 - METHACRYLATE SEAL CONCRETE SURFACES	SQYD	03	636	$221.57	$309.33	$140980.00	04-12-2006	03-0E9004	4		
☑	040169 - METHACRYLATE SEAL CONCRETE SURFACES	SQYD	03	636	$506.69	$707.37	$322392.00	04-12-2006	03-0E9004	5		

uncheck all | check all — cost indexes | legend

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	174.24	**243.26**	Avg No. Units	636
Std Dev. (of Unit Price): ±$	181.24	**253.02**	Rows Selected	5
Weighted Avg.: $	174.25	**243.26**	Rows Returned	5

Previous Estimate

For comparison only, in 2007 the Stevenson Bridge unit price for Furnish Polyester Concrete Overlay (1”) was $3000/CY, the estimated area of repair was 19 CY, which totals to $57k.

Furnish Bridge Deck Treatment Material [Gal]

Bridge length = 40' + 108' + 108' +40' = 296'

Bridge Area = (296' Total Bridge Length from Abut1 to Abut5) (20’ deck width) = 5,920 SF

Material in Gal = 5,920, SF / 90 SF/Gal = 66 Gal

Total Furnish Bridge Deck Treatment, Say 66 Gal

Estimated price = $65/GAL

☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	02	129	$55.00	$55.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	02	129	$84.00	$84.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	02	129	$60.00	$60.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$55.00	$55.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$60.00	$60.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$55.00	$55.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$65.00	$65.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$55.00	$55.00
☑	600047 - FURNISH BRIDGE DECK TREATMENT MATERIAL	GAL	04	75	$70.00	$70.00

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	61.39	**61.39**	Avg No. Units	2470
Std Dev. (of Unit Price): ±$	16.28	**16.28**	Rows Selected	163
Weighted Avg.: $	55.19	**55.19**	Rows Returned	163
Minimum Price/Unit: $	0.60	0.60		
Maximum Price/Unit: $	144.74	144.74		

Core Concrete (6") [LF]

The Core Concrete detail holes in concrete deck to allow placement of concrete for Tie Girder Bolter, Approach Span Bolster, and Pier Cap Bolsters.

Tie Girder Bolter: [(2 cores per bay) (15 bays per quadrant) (4 quadrant) (9"/12 LF)

= 120 EA

Approach Span Bolter: [(8 cores per quadrant this equates to holes drilled every 5 feet) (4 quadrant) (9"/12 LF)

= 32 EA

Pier Cap bolster: [(4 cores per side this equates to holes drilled every 5 feet) (2 sides per pier cap) (3 pier caps) (9"/12 LF)

= 24 EA

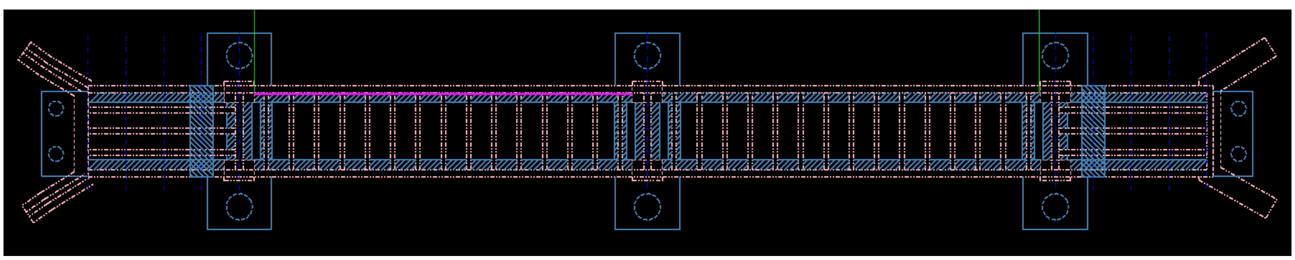
Floor beam bolster: [(4 cores per side this equates to holes drilled every 5 feet) (2 sides per floor beam) (4 floor beams) (9"/12 LF)

= 32 EA

Sub sum of 208 EA at 9" per location = 156 LF

Total Core Concrete (6"), Say $\boxed{156 \text{ LF}}$

Estimated price = $\boxed{\$240/\text{LF}}$



☑	153306 - CORE CONCRETE (6")	LF	12	147	$90.00	$160.29
☑	153306 - CORE CONCRETE (6")	LF	08	120	$190.00	$234.18
☑	153306 - CORE CONCRETE (6")	LF	08	120	$115.00	$141.74
☑	153306 - CORE CONCRETE (6")	LF	08	120	$150.00	$184.88
☑	153306 - CORE CONCRETE (6")	LF	08	120	$268.00	$330.32
☑	153306 - CORE CONCRETE (6")	LF	08	120	$220.00	$271.16
☑	153306 - CORE CONCRETE (6")	LF	08	120	$172.00	$212.00
☑	153306 - CORE CONCRETE (6")	LF	08	120	$147.00	$181.18
☑	153306 - CORE CONCRETE (6")	LF	08	120	$100.00	$123.26
☑	153306 - CORE CONCRETE (6")	LF	08	120	$152.64	$188.14

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	104.71	**239.21**	Avg No. Units	1
Std Dev. (of Unit Price): ±$	67.02	**95.92**	Rows Selected	
Weighted Avg.: $	106.23	**240.50**	Rows Returned	
Minimum Price/Unit: $	40.00	80.15		
Maximum Price/Unit: $	425.00	724.94		

Bridge Removal (Portion) [LS]

Portion of the bridge railing will be required to be removed to retrofit the vertical hanger. The railing railing should be removed in full and reconstructed.

Estimated Lump Sum price = $\boxed{\$50{,}000 \text{ LS}}$

Rock Slope Protection (300 lb, Class IV, Method B) [CY]

RSP volume, Class IV RSP = (2.5 feet thickness) (assumes 30 feet up stream and 30 feet down stream plus 25 feet of the bridge width) (50 feet long transverse per support) (2 supports) / 27

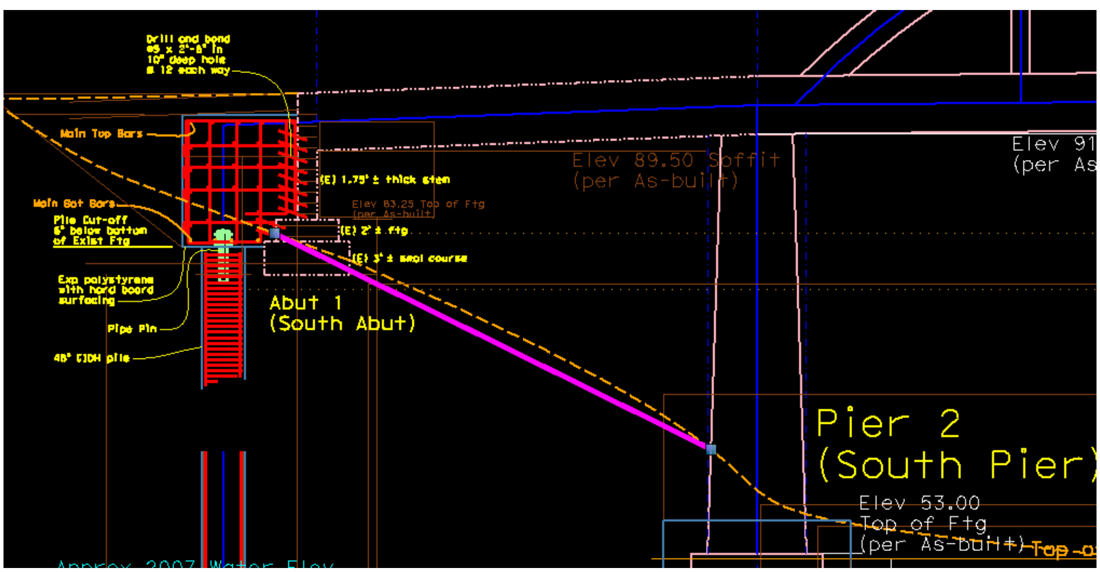
= 787 CY

Include shear key that would be designed during final design, Say $\boxed{\text{800 CY}}$

Estimated price = $\boxed{\text{\$260/CY}}$

5.8 Scour Countermeasures

immediately downstream of the bridge. The minimum RSP class for the existing bridge abutments calculated in accordance with the HEC-23 method is Class III. However, Class IV RSP is recommended based on engineering judgment. Per the *Highway Design Manual,* Class IV RSP at the Project site requires a Class 8 RSP geotextile filter. The minimum RSP layer thickness is 2.5 ft, and detailed RSP calculations are in Appendix D.



☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	06	522	$120.00	$120.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	06	522	$120.00	$120.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	06	522	$129.00	$129.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	06	522	$165.00	$165.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	10	135	$329.00	$329.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	10	135	$750.00	$750.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	01	20	$110.00	$110.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	01	20	$300.00	$300.00
☑	723060 - ROCK SLOPE PROTECTION (300 LB, CLASS IV, METHOD B) (CY)	CY	01	20	$552.75	$552.75

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	256.95	**257.24**	Avg No. Units	252
Std Dev. (of Unit Price): ±$	165.86	**165.78**	Rows Selected	18
Weighted Avg.: $	183.48	**183.77**	Rows Returned	18
Minimum Price/Unit: $	107.00	107.00		
Maximum Price/Unit: $	750.00	750.00		

Rock Slope Protection Fabric (Class 8) [SQYD]

Rock Slope Protection Fabric (Class 8) area = (Assumes 30 feet up stream and 30 feet down stream plus 25 feet of the bridge width) (50 feet long transverse per support) (2 supports)

= 8500 SQFT / 9 = 945 SQYD

Say $\boxed{\text{945 SQYD}}$

Estimated price = $\boxed{\text{\$8/SQYD}}$

☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	09	590	$10.00	$10.00	$5900.00	07-18-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$5.00	$5.00	$3100.00	08-01-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$5.00	$5.00	$3100.00	08-01-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$2.00	$2.00	$1240.00	08-01-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$1.50	$1.50	$930.00	08-01-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$11.00	$11.00	$6820.00	08-01-2017
☑	729011 - ROCK SLOPE PROTECTION FABRIC (CLASS 8)	SQYD	02	620	$8.00	$8.00	$4960.00	08-01-2017

uncheck all | check all

SUMMARY	Unmodified	Adjusted		
Average Price/Unit: $	5.73	**7.28**	Avg No. Units	1099
Std Dev. (of Unit Price): ±$	8.58	**10.57**	Rows Selected	397
Weighted Avg.: $	5.16	**6.53**	Rows Returned	397
Minimum Price/Unit: $	0.95	1.00		
Maximum Price/Unit: $	103.93	136.16		

Miscellaneous Metal (Bridge) [LB]

The miscellaneous metal includes the pipe pin detail that would be installed at the abutment pile head.

Other miscellaneous metal could include architectural treatments that are needed to replace the existing architectural treatments.

For preliminary quantity estimate use a past project example:

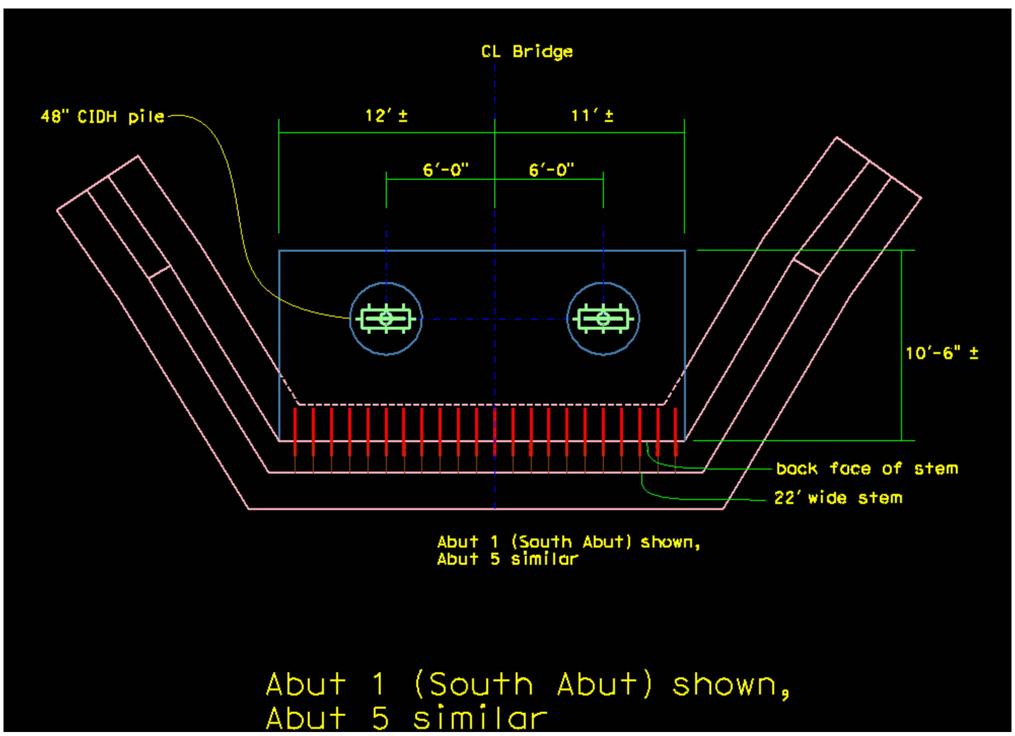
Goleta parkway pipe pin quantity = 909 lbs for 4 pipe pins.

Estimate for Stevenson = (909 lbs) (1.25 factor for special transverse release detail pipe pin) = 1136 lbs

Total Miscellaneous Metal, Say $\boxed{\text{1,200 lbs}}$

Estimated price = $\boxed{\text{\$15/lb}}$

Goleta parkway pipe pin unit price was estimated at $12/lb.



750504	MISCELLANEOUS METAL (PIPE PIN)	LB	909	$ 12.00	$ 10,908.00

Reconstruct Bridge Railing [LF]

Bridge Length = [(40'+108'+108'+40' linearly per side of railing) (2 sides) + (20’ Abut1 WW) (2 side) + (20' Abut5 WW by scale) (2 sides)] = 672 ft

Say $\boxed{672 \text{ LF}}$

The Items should also include concrete and reinforcing steel.

Based on Caltrans bid history, the Reconstruct Bridge Railing is around plus and minus $100/LF. However, for the Stevenson project, reconstructing the bridge railing to match the existing appearance requires specialized forms which are more intricate than typical bridge railing. Therefore, a higher unit price is appropriate. Note: Reconstruction of the bridge railing is also required to fully wrap the FRP full height of the Vertical Hanger regardless of rail condition.

Estimated price = $\boxed{\$650/\text{LF}}$

	Item No. / Description	Unit	Dist	Qty	Unit Price	Adj Price
☑	045104 - RECONSTRUCT BRIDGE RAILING	LF	07	56	$75.00	$157.61
☑	033412 - RECONSTRUCT BRIDGE RAILING	LF	04	150	$83.00	$174.43
☑	043637 - RECONSTRUCT BRIDGE RAILING	LF	02	600	$50.00	$57.38
☑	043637 - RECONSTRUCT BRIDGE RAILING	LF	02	600	$27.00	$30.99
☑	043637 - RECONSTRUCT BRIDGE RAILING	LF	02	600	$35.00	$40.17
☑	043637 - RECONSTRUCT BRIDGE RAILING	LF	02	600	$50.99	$58.52
☑	043637 - RECONSTRUCT BRIDGE RAILING	LF	02	600	$125.00	$143.45

Previous Estimate

For comparison only, in 2007 the Stevenson Bridge unit price for Refinish Bridge Railing was $150/SF, with an estimated area of repair resulting in a unit price of $647/LF.

QUINCY ENGINEERING, INC.

Project Stevenson Road Bridge **Description** Roadway Quantities

Job No.

By A. Mitchell **Date** 1/9/18

SHEET 1 / 1

HMA (areas from CAD)

"S" Sta 40+75.00 → 50+50.00

$A = 30{,}276.0 \ ft^2$

depth = 0.45 ft

$V = 30{,}276.0 \ ft^2 (0.45 \ ft) = 13{,}624.2 \ ft^3$

$$13{,}624.2 \ ft^3 \left(\frac{150 \ lb}{ft^3}\right)\left(\frac{1 \ ton}{2000 \ lb}\right) = \boxed{1{,}021.8 \ tons}$$

AB (areas from CAD)

"S" Sta 40+75.00 → 50+50.00

$A = 30{,}276.0 \ ft^2$

depth = 1.7 ft

$V = 30{,}276.0 \ ft^2 (1.7 \ ft) = 51{,}469.2 \ ft^3$

$\Downarrow$

$\boxed{1{,}906.3 \ CY}$

Tack Coat 0.45' HMA (two lifts, 0.25' + 0.2', apply tack coat between layers)

$A = 30{,}276.0 \ ft^2 \Longrightarrow \underline{3{,}364 \ SY}$

$$3{,}364 \ SY \left(\frac{0.03 \ gal}{SY}\right)\left(\frac{1 \ ton}{240 \ gal}\right) = \boxed{0.42 \ ton}$$

Bonded Fiber Matrix (areas in CAD)

$$\left.\begin{array}{llll} \text{south of bridge} & Rt & : & 10{,}119 \ ft^2 \\ \text{" " "} & Lt & : & 12{,}395 \ ft^2 \\ \text{north " "} & Rt & : & 1{,}320 \ ft^2 \\ \text{" " "} & Lt & : & 609 \ ft^2 \end{array}\right\} \boxed{24{,}443 \ ft^2}$$

Baseline Station	CUT Area	CUT Volume	FILL Area	FILL Volume
40+75.00	44.43	0	0.02	0
40+80.00	44.75	8.26	0.08	0.01
41+00.00	45.41	33.39	0.35	0.16
41+20.00	46.29	33.96	0.67	0.38
41+40.00	47.56	34.76	1.08	0.65
41+60.00	46.31	34.77	1.45	0.94
41+80.00	46.07	34.22	1.8	1.2
42+00.00	46.37	34.24	1.78	1.32
42+20.00	47.46	34.75	1.07	1.05
42+40.00	48.54	35.56	0.49	0.58
42+60.00	49.67	36.38	0.15	0.24
42+80.00	48.52	36.37	1.29	0.53
43+00.00	44.28	34.37	10.86	4.5
43+20.00	34.92	29.33	12.96	8.82
43+40.00	24.57	22.03	10.37	8.64
43+60.00	18.19	15.84	10.35	7.67
43+80.00	18.76	13.68	19.27	10.97
44+00.00	23.35	15.6	27.21	17.21
44+20.00	23.18	17.24	19.38	17.25
44+40.00	24.06	17.5	17.37	13.61
44+60.00	17.82	15.51	18.07	13.13
44+80.00	17.68	13.15	21.04	14.49
45+00.00	16.63	12.71	22.6	16.16
45+20.00	16.46	12.26	22.58	16.74
45+40.00	16.43	12.18	22.71	16.77
45+60.00	12.62	10.76	22.15	16.62
45+80.00	9.39	8.15	19.88	15.57
46+00.00	13	8.29	22.86	15.83
46+20.00	4.4	6.44	20.73	16.14
46+40.00	9.37	5.1	23.31	16.31
46+60.00	0.49	3.65	22.29	16.89
46+80.00	3.1	1.33	25.31	17.63
47+00.00	0.16	1.21	23.9	18.22
47+20.00	4.71	1.8	30.59	20.18
47+40.00	1.36	2.25	27.15	21.39
47+60.00	6.03	2.74	33.32	22.4
47+80.00	6.3	4.56	32.51	24.38
48+00.00	6.85	4.87	33.58	24.48
48+20.00	9.06	5.89	33.98	25.02
48+40.00	7.3	6.06	33.95	25.16
48+60.00	4.82	4.49	38.13	26.7
48+80.00	5.6	3.86	41.94	29.66
49+00.00	1.78	2.73	46.33	32.69
49+20.00	1.29	1.14	55.71	37.79
49+40.00	0.22	0.56	66.12	45.12
49+60.00	0	0.08	96.96	60.4
49+80.00	0	0	110.67	76.9
50+00.00	100.3	37.15	3.25	42.19
50+20.00	109.46	77.69	0	1.2
50+40.00	53.19	60.24	42.78	15.85
50+50.00	40.75	17.4	419.7	85.65
Grand Total:		866.49		923.37

Baseline Station	CUT Area	CUT Volume	FILL Area	FILL Volume
53+47.01	0.1	0	255.94	0
53+60.00	1.68	0.43	42.68	71.86
53+80.00	2.32	1.48	15.07	21.39
54+00.00	2.76	1.88	8.54	8.75
54+20.00	3.34	2.26	6.61	5.61
54+22.01	0	0.12	0	0.25
Grand Total:		6.17		107.85

Roadway Excavation:	872.66	CY
Embankment:	1031.22	CY
Imported Borrow:	158.56	CY

Appendix C - Basis of Design and Design Criteria Memorandum

Design Criteria Memorandum – Summary Table

Project Description: Rehabilitate the bridge and realign south approach road (Stevenson Bridge Road).

Client/Agency: Solano County
Facility Owner: Solano County
Project Number: S31-200

Project Name: Stevenson Bridge Rehabilitation Project
Date: 6/27/16

	GENERAL PROJECT INFORMATION
Current ADT/ Future ADT	789 (2008) - BIRIS 1518 (2035) - BIRIS
Terrain	Level
Street Type/ Functional Classification	Minor Collector (Per Caltrans CRS Map 6J)
Plans to change Classification in the Future	No
Designated Bicycle or Pedestrian Facility? Address ADA requirements	Bicyclists are to be considered during design.
Construction Year/Design Year (20-years from construction)	Construction Year: 2017/2018 Design Year: 2037/2038
Funding Source	HBP and County

ROAD A(1)	***Stevenson Bridge Road***

Criteria	Local Standards (Solano County)	AASHTO Greenbook Guidelines (2011)	Proposed Standard	Comments (Note here if a design exception is needed)
Design Speed	Refers to AASHTO (pg. 4, Sec. 1-2.2)	With ADT between 400-2000 and level terrain, DS=50 mph (pg. 6-2, Table 6-1)	35 mph	Per kickoff meeting w/ Solano County, the desired DS (approved by Caltrans) is 25 mph. – **DESIGN EXCEPTION REQUIRED**
Traffic Index	Not enough information to use Figure 1 below. (pg. 19, Figure 1)	No guidance.	7	TI=7 given to QEI by the County on 6/13/16 (email).

-

Design Criteria Memorandum – Summary Table

Project Description: Rehabilitate the bridge and realign south approach road (Stevenson Bridge Road).

Client/Agency: Solano County
Facility Owner: Solano County
Project Number: S31-200

Project Name: Stevenson Bridge Rehabilitation Project
Date: 6/27/16

Criteria	Local Standards (Solano County)	AASHTO Greenbook Guidelines (2011)	Proposed Standard	Comments (Note here if a design exception is needed)
R value	In lieu of testing, a design R-value of 5 may be used. (pg. 6, Sec. 1-2.8)	No guidance.	5 (Geotech to test)	
Structural Section	a) 3"AC/9" AB b) 6" AC (pg. 6, Sec. 1-2.8)	No guidance.	TI=7: 0.35' HMA/1.25' AB TI=8: 0.40' HMA/1.48' AB TI=9: 0.45' HMA/1.71' AB	The County to select a structural section based on a variable TI value. (See options to the left)
Lane Width	12' (pg. 5, Sec. 1-2.7)	11' lanes (pg. 6-6, Table 6-5)	12'	Per County Standards
Outside Shoulder Width	4' for "enhanced width roads" (pg. 5, Sec. 1-2.7)	6' (pg. 6-6, Table 6-5)	4' paved shoulders	Per Kickoff Meeting (for cyclists), the County would like paved shoulders
Min Width of Traveled Way	With ADT between 751-4000, DS ≥ 30 mph, min. width is 24' traveled way (pg. 5, Sec. 1-2.7)	22' (pg. 6-6, Table 6-5)	24'	
Distance from Edge of Shoulder to Hinge Point	4' graded shoulders (pg. 5, Sec. 1-2.7)	No guidance.	4' graded shoulder	Figure 3 of the Solano Co. standards, show the graded shoulders with a 5% slope.
Side Slopes (Cut/Fill)	2:1 or flatter (pg. 21, Figure 3)	No guidance.	2:1	
Min. Stopping Sight Distance	Subject to the Director's requirements. (pg. 5, Sec. 1-2.6)	With a DS=35 mph, SSD=250' (pg. 6-4, Table 6-3)	250' (DS=35mph)	

QUINCY ENGINEERING

-

Design Criteria Memorandum – Summary Table

Project Description: Rehabilitate the bridge and realign south approach road (Stevenson Bridge Road).

Client/Agency: Solano County
Facility Owner: Solano County
Project Number: S31-200

Project Name: Stevenson Bridge Rehabilitation Project
Date: 6/27/16

Criteria	Local Standards (Solano County)	AASHTO Greenbook Guidelines (2011)	Proposed Standard	Comments (Note here if a design exception is needed)
Vertical Grades (Min/Max)	Refers to AASHTO (pg. 4-5, Sec. 1-2.4)	Min grade = 0.50% (pg. 3-119) With a DS=35 mph and Level Terrain, Max Grade is 7% (pg. 6-3, Table 6-2)	Min: 0.50% Max: 7%	
Min. K value for: CREST SAG	No guidance.	With DS=35 mph: Sag K_{min}: 49 Crest K_{min}: 29 (pg. 6-4, Table 6-3)	Sag K_{min}: 49 Crest K_{min}: 29 PSD Crest K_{min}: 108	
Min. Vertical Curve Length	No guidance.	L_{min}=3V=3x25=75' (pg. 3-153)	75'	
Min. Horizontal Curve Radius	No guidance.	With e_{max}=6%, R_{min}=144' (pg. 3-45, Table 3-9)	340' (DS=35 mph)	
Maximum Superelevation (e_{max})	No guidance.	E_{max}=6% (pg. 3-31)	6% e_{max}	6% e_{max} is appropriate for the project's rural setting.
Normal Cross Slope	2% (pg. 21, Figure 3)	1.5-2% (pg. 6.-3)	2%	
Pavement Corner Radii	10' for driveway (pg. 25, Figure 7)	Based on Design Vehicle (pg. 9-58, Table 9-15)	TBD	
Minimum Corner Sight Distance at Intersections	No guidance.	With DS=35mph: 165' (pg. 9-33, Table 9-3)	165' (DS=35 mph)	

-

Design Criteria Memorandum – Summary Table

Project Description: Rehabilitate the bridge and realign south approach road (Stevenson Bridge Road).

Client/Agency: Solano County
Facility Owner: Solano County
Project Number: S31-200

Project Name: Stevenson Bridge Rehabilitation Project
Date: 6/27/16

Criteria	Local Standards (Solano County)	AASHTO Greenbook Guidelines (2011)	Proposed Standard	Comments (Note here if a design exception is needed)
Clear Zone Width	Subject to the Director's requirements. (pg. 5, Sec. 1-2.6) For Utility Poles, horizontal clear distance shall be 8'. (pg. 7, Sec. 1-2.14)	With DS ≤ 40 mph, and ADT between 1500-6000, CRZ =14-16' (2011 AASHTO RDG, pg. 3-3, Table 3-1)	14-16'	
Minimum Right of Way Width	For ADT between 751-4000, ROW width shall be 70' (pg. 5, Sec. 1-2.7)	No guidance.	70'	
Drainage Design Storm	N/A	N/A	N/A	No anticipated drainage improvements.
Design Vehicle	No guidance.	No guidance.	TBD	

Additional Project Information:

1. Traffic Handling?
    - Road construction to be completed under traffic control.
    - No road closures or detours anticipated for the roadway work.
    - Detour will be required for bridge rehabilitation.

2. Are there any obstacles (both existing and future) which may affect the stopping sight distance?
    - Existing condition: Existing orchard may block the sight distance to construction zone.

3. What is the operating speed of the facility?
    - 25 mph (2 hairpin turns approaching Stevenson Bridge)

4. Any issues that affect alignment? Such as right of way, environmentally sensitive areas, existing infrastructure to avoid?
   - APE Limits have already been delineated. Project needs to stay within these limits. A majority of the new roadway alignment will be going through an existing orchard. ROW will need to be obtained prior to construction.

5. Does the client have special requests or considerations they want addressed?
   - Paved Shoulders. Per Solano County, this falls under the classification of an “Enhanced Width Road.”

6. Room for standard flared bridge approach and departure railing? Length and width
   - Yes, depending on alignment, there should be adequate areas for standard flared bridge approach railing on the south side. On the north side, there may not be enough width and an in-line terminal system may need to be utilized. On the south end, the pavement will conforming to a narrower pavement width at the Stevenson Bridge. Approach railings will not be parallel with the bridge ends or wingwalls. More than likely, a concrete block will have to be constructed for the approach railing to anchor onto.

7. Distance to approach road/intersections or driveways from bridge?
   - There is an existing driveway that intersects Stevenson Bridge Road approximately 550’ east of the bridge. This driveway will have to be extended to meet up with the new Stevenson Bridge Road alignment. Depending on roadway alignment, the location of the “driveway extension” will vary.

8. Street lighting required? Standards?
   - No. Only required in areas designated as “RE-1”. (Solano Co. Stds. pg. 11, Sec. 1-5.2)

9. Temporary and Permanent Storm Water Treatment – are there agency specific BMP’s? Is there a Phase 1 or Phase 2 MS4 Permit (provide reference)?
   - Solano County is determining if the BASMAA guidelines apply for post construction stormwater treatment.

-

Design Criteria Memorandum – Summary Table

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Client/Agency: Solano County
Facility Owner: Solano County
Project Number: S31-200

Project Name: Stevenson Bridge Rehabilitation Project
Date: 6/27/16

Submitted By:

________________________ Date ________
Design Engineer

Approvals:

Quincy Engineering, Inc.

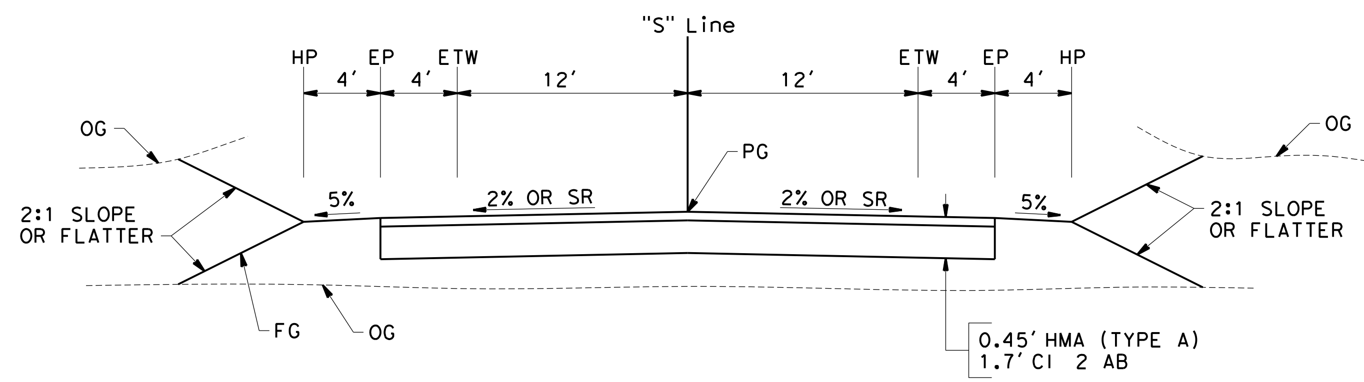
________________________ Date ________
Project Engineer

__________________________ Date _______
Project Manager

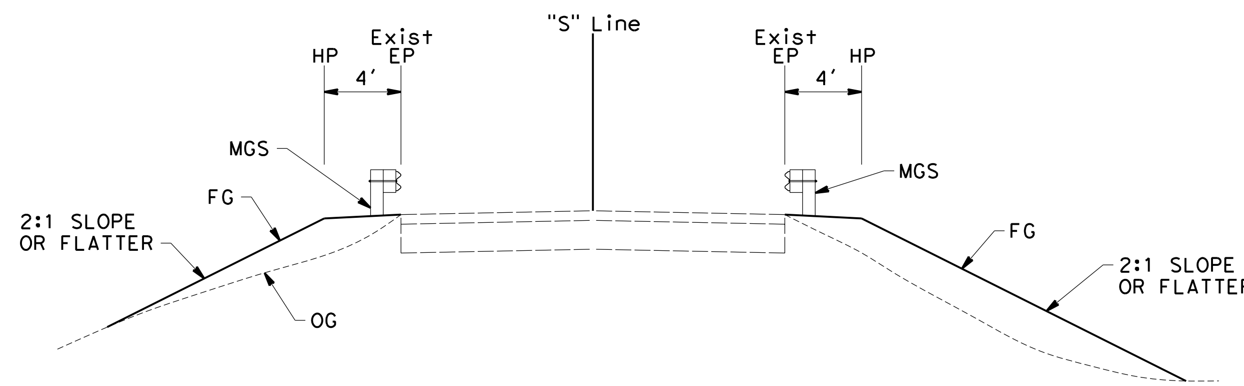
___________________________Date ____________
Principal in Charge

Revised___________________________

Appendix D - 30% Roadway Plans



STEVENSON BRIDGE ROAD
(SOUTH OF BRIDGE)
No Scale

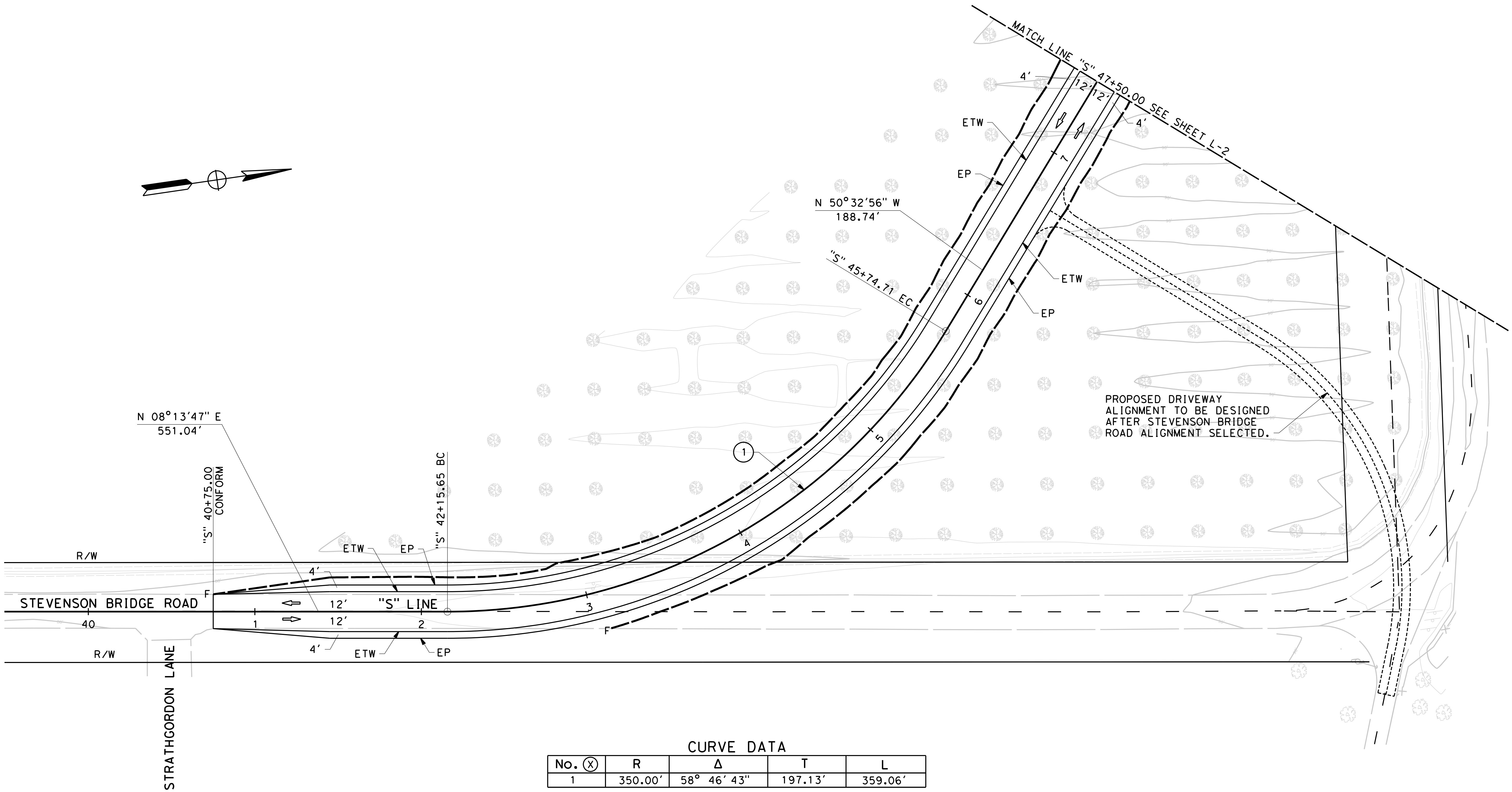


STEVENSON BRIDGE ROAD
(NORTH OF BRIDGE)
No Scale

REVISIONS				FIELD BOOK NO.	SCALE					
NO.	DESCRIPTION	APPROVED BY	DATE		HORIZONTAL: AS NOTED VERTICAL: AS NOTED	DRAWN BY: CHECKED BY: SUBMITTED R.C.E. No.	SOLANO COUNTY TRANSPORTATION DEPARTMENT 333 SUNSET AVE. SUITE 230 SUISUN CITY CA 94585 TEL: (707) 421-6069 FAX: (707) 429-2894	APPROVED BY: DATE:	STEVENSON ROAD BRIDGE 35 MPH ALIGNMENT TYPICAL CROSS SECTION	DATE x/xx/xx SHEET X OF X DWG

NOTE:

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA,
SEE RIGHT OF WAY RECORD MAPS AT THE COUNTY OFFICE.



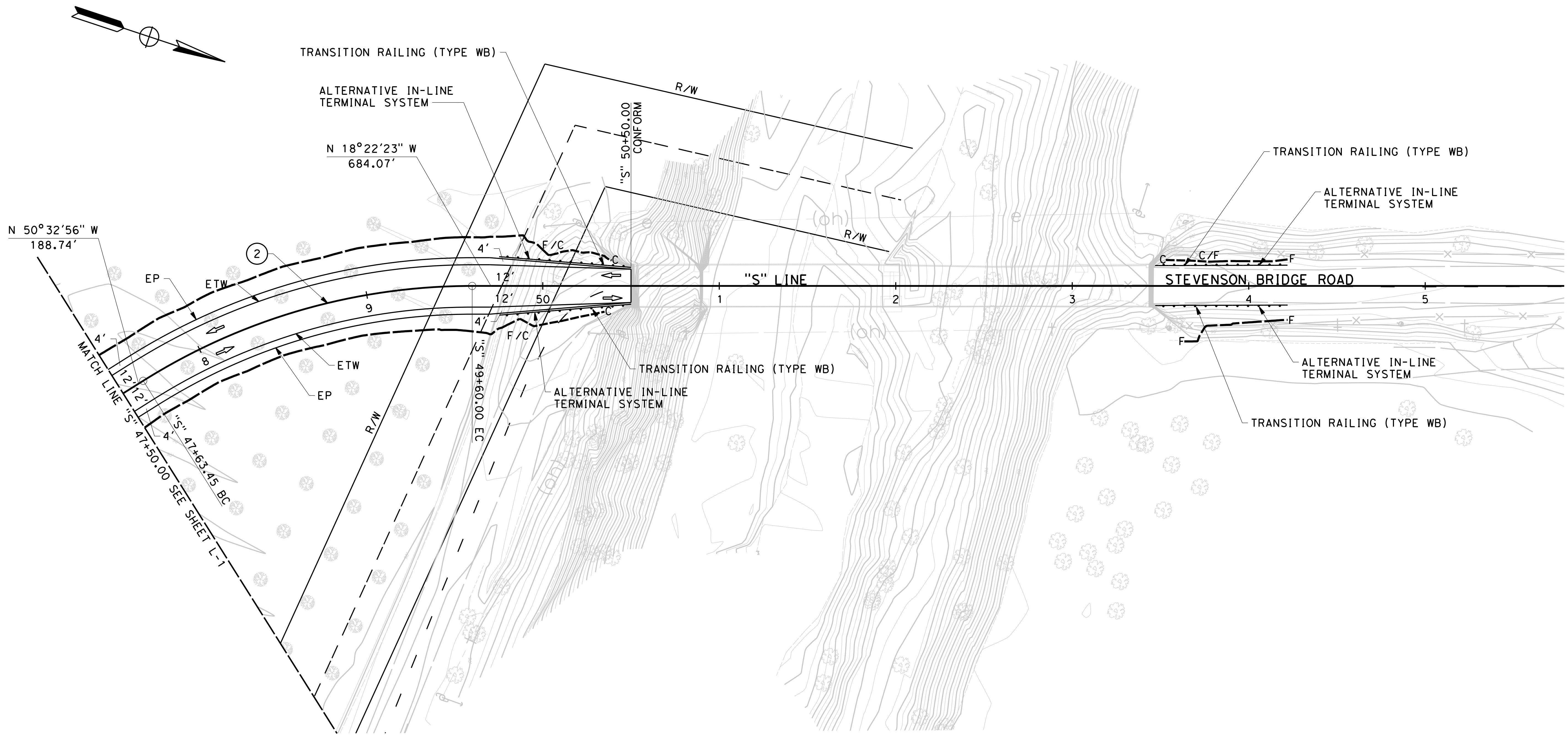
CURVE DATA

No. (X)	R	Δ	T	L
1	350.00'	58° 46' 43"	197.13'	359.06'

REVISIONS				FIELD BOOK NO.	SCALE					
NO.	DESCRIPTION	APPROVED BY	DATE		HORIZONTAL: 1"=30' VERTICAL: NONE	DRAWN BY: CHECKED BY: SUBMITTED R.C.E. No.	SOLANO COUNTY TRANSPORTATION DEPARTMENT 333 SUNSET AVE. SUITE 230 SUISUN CITY CA 94585 TEL: (707) 421-6069 FAX: (707) 429-2894	APPROVED BY: DATE:	STEVENSON ROAD BRIDGE 35 MPH ALIGNMENT LAYOUT L-1	DATE x/xx/xx SHEET X OF X DWG

NOTE:

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA,
SEE RIGHT OF WAY RECORD MAPS AT THE COUNTY OFFICE.



CURVE DATA

No. ⓧ	R	Δ	T	L
2	350.00'	32° 10' 33"	100.94'	196.55'

REVISIONS			
NO.	DESCRIPTION	APPROVED BY	DATE

FIELD BOOK NO.

SCALE
HORIZONTAL: AS NOTED
VERTICAL: AS NOTED

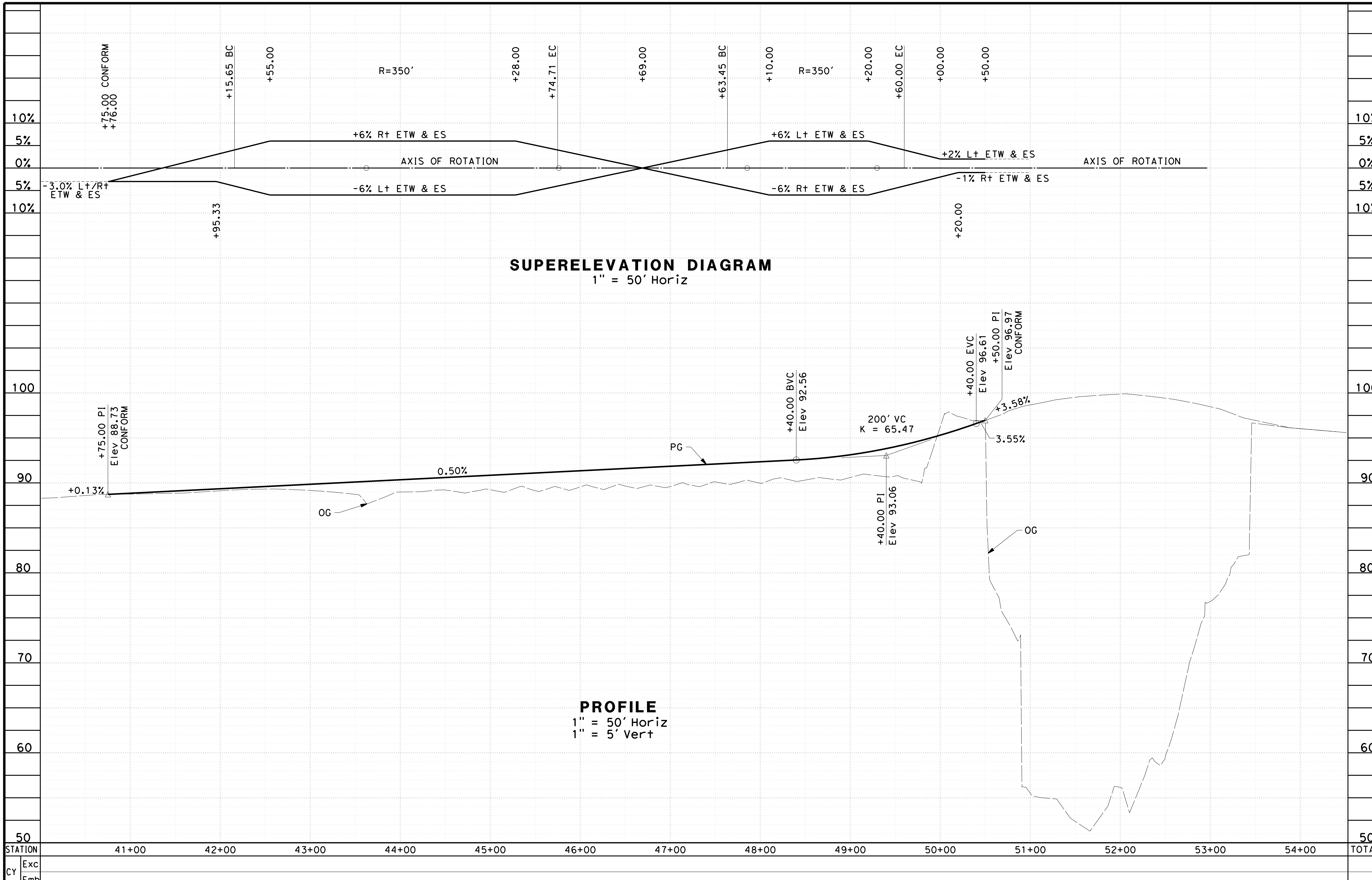
DRAWN BY: CHECKED BY:
SUBMITTED R.C.E. No.

SOLANO COUNTY
TRANSPORTATION DEPARTMENT
333 SUNSET AVE. SUITE 230
SUISUN CITY CA 94585
TEL: (707) 421-6069 FAX: (707) 429-2894

APPROVED BY:
DATE:

STEVENSON ROAD BRIDGE
35 MPH ALIGNMENT
LAYOUT L-2

DATE x/xx/xx
SHEET X OF X
DWG



REVISIONS			
NO.	DESCRIPTION	APPROVED BY	DATE

FIELD BOOK NO.

SCALE
HORIZONTAL: AS NOTED
VERTICAL: AS NOTED

DRAWN BY: CHECKED BY:
SUBMITTED R.C.E. No.

SOLANO COUNTY
TRANSPORTATION DEPARTMENT
333 SUNSET AVE. SUITE 230
SUISUN CITY CA 94585
TEL: (707) 421-6069 FAX: (707) 429-2894

APPROVED BY:
DATE:

STEVENSON ROAD BRIDGE
35 MPH ALIGNMENT
PROFILE AND SUPERELEVATION DIAGRAM

DATE x/xx/xx
SHEET X OF X
DWG

Appendix E - Draft Foundation Report

DRAFT FOUNDATION REPORT

STEVENSON BRIDGE ROAD BRIDGE OVER PUTAH CREEK
SOLANO COUNTY AND YOLO COUNTY, CALIFORNIA

31 OCTOBER 2016

Prepared for:

Quincy Engineering, Inc.
11017 Cobblerock Drive, #100
Rancho Cordova, California 95670

Prepared by:

Cal Engineering & Geology, Inc.
1870 Olympic Boulevard, Suite 100
Walnut Creek, California 94596

Chris Hockett, P.E., G.E.
Associate Engineer

Dave Burger, P.G., E.G.
Senior Geologist

Mehal Vitthal, E.I.T.
Project Engineer

Reviewed by:

Phil Gregory, P.E., G.E.
Senior Principal Engineer

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

This Foundation Report (FR) presents the results of geotechnical subsurface exploration for the planned repair of the Stevenson Bridge over Putah Creek located along Stevenson Bridge Road at the Solano County / Yolo County boundary. The purpose of this FR is to document the subsurface conditions and provide analyses of anticipated site conditions as they pertain to design and construction of the bridge repair and roadway realignment.

2.0 SCOPE OF WORK

The scope of work completed to prepare this FR consisted of the following:

- Obtained copies of the available published geologic data and maps for the site and vicinity;
- Performed a site visit to note the surface geology and topography, distinguish site accessibility and construction constraints, photo-document the bridge foundation improvement locations, and mark boundary limits for underground utility locating using white paint;
- Contacted USA North (USA) a minimum of 48-hours prior to performing subsurface drilling operations to have USA alert utility subscribers to mark their underground utilities;
- Obtained drilling and encroachment permits from Solano and Yolo Counties;
- Provided and implemented traffic control in coordination with the Solano and Yolo Counties;
- Drilled and sampled three (3) geotechnical test borings, one at each of the existing pier locations.
- Completed laboratory testing on selected soil samples collected during drilling operations to refine/determine soil classifications and engineering and physical properties of the soil;
- Analyzed the collected geotechnical data and developed design recommendations;
- Created log of test boring (LOTB) sheets to present the subsurface findings in plan and profile orientation; and
- Prepared this FR summarizing the findings, recommendations, and analyses.

3.0 PROJECT DESCRIPTION

The project site is located along Stevenson Bridge Road at Putah Creek on the boundary of Solano and Yolo Counties (Appendix A). This project consists of planned rehabilitation to the Stevenson Bridge Road Bridge (Bridge Number 23C-0092), which traverses Putah Creek to connect Solano County at the Southern Abutment (Abutment 1) and Yolo County at the Northern Abutment (Abutment 5). The existing bridge is a reinforced concrete through-tied-arch bridge approximately 24.5 feet in width and spanning 296 feet across Putah Creek. The bridge is supported on the abutments at the north and south ends and three intermediate piers. The bridge was built in 1923 and is founded on spread footings at the abutments and timber and concrete piles at the piers.

The existing structure is in a deteriorated condition and rated as structurally deficient by Caltrans. The current load capacity of critical structural members do not meet the demands induced by the design seismic event (TRC Imbsen, 2007). Structural deterioration is visually evident on the current structure with large cracks, spalling of concrete, and exposed steel reinforcement. The planned project is to retrofit and rehabilitate the existing bridge to restore structural integrity of the bridge and address concerns of public safety. It is anticipated that rehabilitation will include modifications to the existing foundations that will include construction of CIDH piles adjacent to the existing timber/concrete piles at each bridge pier. In addition, CIDH piles are proposed to supplement the spread footings at each of the abutments.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 TOPOGRAPHY AND GEOLOGY

The topography in the region is essentially flat with the exception of areas immediately adjacent to the Putah Creek channel. The creek channel is approximately 45 feet deep in the vicinity of the bridge with the southern creek bank steeply inclined and the northern creek bank moderately to steeply inclined in the vicinity of the project area.

The project site is situated within the Great Valley Geomorphic Province near the western boundary (Jennings, 1977). This portion of Solano and Yolo Counties is comprised of primarily marine and non-marine sediments deposited within the late Cenozoic Era.

The generalized geology of the greater Dixon/Davis area has been mapped by a number of geologists (Jennings, 1977), (Graymer, 2002), and (Graymer, 2006). Each of the maps by these geologists indicate that the project site is underlain by Holocene age alluvium. Graymer, 2002, indicates the project site is underlain by Quaternary age Holocene natural levee deposits (Appendix B). To the north and south of the project area, Graymer indicates Quaternary age

Holocene alluvial fan deposits. This mapping is consistent with the materials encountered in our subsurface explorations.

It should be noted that the site is located approximately 1,700 feet to the northwest of the East Valley Fault, 3 miles to the northwest of the West Valley Fault, and 8 miles north of the Midland Fault (Wagner et. al., 1981). Additionally, numerous small segments of Historic and Quaternary age faults related to the movements of the Vaca and Bennett Valley Fault Zones have been identified approximately 13 and 20 miles to the southeast of the project site, respectively (Graymer, 2006).

4.2 TYPES OF SOIL

The surficial soils in the vicinity of the project site has been mapped by the United States Department of Agriculture National Resource Conservation Service. The site has been classified with four soil types (Appendix C). The Solano County soils have been classified as belonging to Riverwash within the Putah Creek channel and the Yolo Loam for 0 to 4 percent slopes to the south. The Yolo County soils have been classified as belonging to Riverwash within the Putah Creek channel and Yolo Silt Loam for 0 to 2 percent slopes to the north (NRCS, 2016).

4.2.1 Solano County Soils

Riverwash soils are excessively well-drained, the frequency of flooding is found to be frequent, considered non-plastic, the risk of corrosion of uncoated steel and concrete is low, and they are found in channels. The Yolo Loam soils are well-drained, the frequency of flooding is found to be rare, runoff class is low, plasticity index ranging between 6 and 19 percent in the upper 60 inches, risk of corrosion of uncoated steel and concrete is low, and are found in alluvial fans.

4.2.2 Yolo County Soils

Riverwash soils are excessively drained, the frequency of flooding is found to be frequent, runoff class is negligible, considered non-plastic to a plasticity index of 2 percent in the upper 60 inches, the risk of corrosion of uncoated steel and concrete is nil, and are found in channels on streams. The Yolo Loam soils are well-drained, the frequency of flooding is found to be rare, runoff class is low, plasticity index ranging between 9 and 30 percent in the upper 65 inches, risk of corrosion of uncoated steel and concrete is low, and are found in alluvial fans and flood plains.

4.3 PERTINENT SOIL CONDITIONS OR GEOLOGIC HAZARDS

The U.S. Geological Survey has mapped the Quaternary deposits and liquefaction susceptibility of nine San Francisco Bay Area counties. The southern portion of the project is located within Solano County which has been mapped as having a moderate liquefaction susceptibility

(Knudsen, 2000). Knudsen also indicates that the sediments deposited within creek channels have a very high liquefaction susceptibility. The area to the north of Putah Creek within Yolo County has not been mapped in the above mentioned study. However, given the similar depositional environments, it is likely that the northern portion of the project area has a moderate liquefaction susceptibility.

4.3.1 Landslides and Creek Bank Stability

We were unable to locate regional landslide maps of the area by the U.S. Geological Survey and the California Geological Survey. However, based on observations during the subsurface exploration operation, we did not observe landslide features at the project site. Since the site is essentially flat outside the creek channel, the potential for landsliding is low. However, the creek banks are steeply inclined and upwards of 45 feet tall. Therefore, it is our opinion that the hazard of landslides impacting the planned improvements within the creek channel should be considered to be moderate near the northern and southern bridge abutments.

Considering the proximity to the steep creek banks along the northern and southern limits of the project site, it is our opinion that the long-term potential of landslides developing along the creek bank should be considered moderate.

The segment of Putah Creek in the vicinity of the project area, shallow slump failures were not observed. However erosion features including rills up to 6-inches deep at various locations along both banks, calving of up to 1 foot of material near the creek invert, and erosion of the sediments exposing the existing scour protection at the existing pile caps were observed.

4.3.2 Loose Sands, Gravels, and Cobbles

Coarse sands, rounded fine and course gravel, and isolated cobbles were encountered in the geotechnical borings. The presence of these materials resulted in drilling fluid loss and caving during the subsurface exploration program. Casing was required to maintain drilling fluid circulation and to prevent caving. The depth of casing is shown on the boring logs in Appendix D.

4.4 GROUNDWATER ELEVATION

Groundwater was encountered during the subsurface exploration drilling operation of Boring B-2 at a depth of 3 feet below the ground surface. This corresponds to the approximate normal water surface elevation of Putah Creek. Groundwater was not measured in Boring B-1 and B-3 due to rotary wash methods.

Kleinfelder (2006) measured groundwater at a depth of 46-½ feet below the ground surface in their Boring B-1 and at a depth of 50 feet below the ground surface in their Boring B-2.

Groundwater levels can vary over time in response to environmental/seasonal and land use changes. For this reason, groundwater levels at the time of construction or in the future could differ from those encountered at the time of the subsurface exploration.

5.0 FIELD INVESTIGATION AND TESTING PROGRAM

5.1 PREVIOUS INVESTIGATION

Kleinfelder completed a geotechnical investigation at the project site and presented findings in their report titled, "Geotechnical Investigation Report, Existing Stevenson Bridge, Stevenson Bridge Road at Putah Creek" dated April 28, 2006 (Kleinfelder, 2006). Kleinfelder's subsurface exploration program included two borings advanced to 101.5 feet below the existing roadway surface near each abutment of the bridge. The Kleinfelder boring logs are included in Appendix E.

5.2 SUBSURFACE EXPLORATION

Three test borings for this foundation report were advanced and sampled between 12 September 2016 and 20 October 2016. The borings were completed under subcontract to Cal Engineering & Geology (CE&G) by Woodward Drilling of Rio Vista, California. Test boring locations and depths were selected based on the anticipated positioning and lengths of the cast-in-drilled-hole (CIDH) concrete piles. The borings were located as close as possible to the anticipated CIDH concrete piles. The locations were adjusted in the field to account for site access constraints (sloping ground, trees). The final locations were measured off established site features and marked upon completion.

The test borings were drilled to the following depths below grade:

- Boring R-16-001(B-1): 121.0 feet,
- Boring R-16-002(B-2): 129.5 feet,
- Boring R-16-003(B-3): 139.0 feet.

The depths were selected to gather subsurface data to at least 20 feet below the anticipated pile tip elevations in conformance with AASHTO Bridge Design Specifications Guidelines. (AASHTO, 2012). Test borings R-16-001 (B-1) and R-16-002 (B-2) were drilled and sampled using a Mobile B57 track-mounted drilling rig using a 3-7/8-inch diameter bit rotary wash recirculation system. Access to boring B-1 and B-2 was provided by lowering the drill rig off the existing bridge deck using a crane. Woodward Drilling, Inc. subcontracted the crane using Summit Crane of Vacaville, California. Lowering and raising of the drilling rig and accessory equipment required traffic control to close the bridge and provide an approximate 12-mile long detour. Traffic control was provided under subcontract to CE&G by Traffic Control Pros of Concord, California.

Boring R-16-003 (B-3) was drilled and sampled using a Mobile B57 truck-mounted drilling rig using a 3-7/8-inch diameter bit rotary wash recirculation system. Access to the boring location was provided through a University of California Davis managed access road along the eastern side of Stevenson Bridge Road to the north of the bridge. During the mobilization to the boring location, a biologist subcontracted through Solano County provided observations and recommendations while work was being performed in close proximity to sensitive plant and animal species.

The sampling protocol was determined based on geologic conditions and by materials encountered during the drilling operation. The materials encountered in the borings were logged in the field by a CE&G senior engineering geologist and senior geotechnical engineer. The soils were classified in the field and office using the Caltrans 2010 Soil and Rock Logging, Classification, and Presentation Manual with the 2015 Errata (Caltrans, 2010). The soils were classified in the laboratory according to the Unified Soil Classification System (USCS) (ASTM D2487).

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- California Modified (CM) Sampler; 3.0 inch outer diameter (O.D.), 2.5 inch inner diameter (I.D.) (ASTM D1586)

- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0 inch O.D., 1.375 inch I.D. (ASTM D1586)

The samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound automatic trip-hammer dropping 30-inches in general conformance with ASTM guidelines (ASTM D6066). The number of blows required to drive the SPT or CM sampler 6-inches was recorded for each sample. In addition, a pocket penetrometer was utilized on appropriate fine grain samples obtained. The blow counts included on the boring logs are uncorrected and represent the field values. The results are included on the log of test boring (LOTB) in Appendix F.

Upon completion of drilling activities, the borings were backfilled to the ground surface with via tremie displacement methods using neat cement grout in accordance with Solano County and Yolo County well drilling permit requirements under observation of their inspectors.

Material spoils and drilling fluid obtained during the drilling and borehole backfilling operations were collected in 55-gallon drums. Woodward Drilling collected the drums and off hauled them for contamination testing and disposal.

5.3 IN SITU TESTING

In situ geotechnical testing completed for this study was limited to Standard Penetration Testing (SPT) (ASTM D1586) sampling. An efficiency rating for the autotrip hammer was provided to CE&G to correlate the field blow count values. Pocket penetrometer tests were completed in appropriate fine grain samples obtained from each boring.

6.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples recovered from the borings. Laboratory tests include: Moisture-Density (ASTM D2216), Atterberg Limits (ASTM D4318), Sieve Analysis (ASTM D422), Minimum Resistivity (Caltrans TM 643), Chlorides (Caltrans TM 422), Sulfate (Caltrans TM 417), pH (Caltrans TM 643), Triaxial Compression Consolidated-Undrained Staged with Pore Pressure (ASTM D4767), Triaxial Compression Unconsolidated - Undrained (ASTM D2850).

Total wet densities ranged from [X to X] pcf for granular soils and between [X to X] pcf for fine grained soil encountered in the field borings.

Atterberg limits tests were completed on select samples. The results are summarized in the table below.

Boring	Sample	Depth (ft)	USCS Soil Classification	LL (%)	PL (%)	PI (%)
B-1	1-6	9.5	ML			
B-2	2-8	14.5	MH			
B-3	3-14	28.0	CL			

Sieve analyses were performed on soil samples from the borings. The results are summarized in the table below.

Boring	Sample	Depth (ft)	USCS Soil Classification	Gravel (%)	Sand (%)	< #200 (%)
B-1	1-3	6.0				
B-1	1-6	9.5				
B-1	1-9	14.5				
B-1	1-11	19.5				
B-1	1-22	59.5				
B-1	1-26	79.5				
B-2	2-2	6.5				
B-2	2-8	14.5				
B-2	2-11	19.5				
B-2	2-22	39.5				
B-3	3-12	23.0				
B-3	3-13	26.5				
B-3	3-14	28				
B-3	3-18	35.5				

Triaxial Compression tests were conducted on selected samples from the geotechnical exploration borings. The results of the test are summarized in the table below.

Boring	Sample	Depth (ft)	USCS Soil Classification	Triaxial Test*	Friction Angle, ϕ (Deg.)	Cohesion Intercept, C (psf)
B-1	1-21	59.0	ML	CU w/ PP		
B-2	2-21	39.0	SP	CU w/ PP		
B-2	2-25	49.0	CL	UU		
B-2	2-31	79.0	ML	CU		
B-3	3-22	42.5	CL	UU		
B-3	3-27	52.0	MH	UU		

* CU – Consolidated Undrained; UU – Unconsolidated Undrained; PP – Pore Preasure

Laboratory test results are presented in Appendix G.

7.0 SCOUR EVALUATION

Evaluation of scour potential was completed by WRECO as part of the project scour and hydraulics analyses and is presented in their report titled, [INSERT WRECO REPORT TITLE] dated [MONTH YEAR].

8.0 CORROSION EVALUATION

The principle cause of deterioration of concrete in foundations is attack by sulfates in soil and groundwater. Chlorides present in the environment do not represent a hazard to concrete, but can cause corrosion to reinforcing steel and other buried metals. Corrosion of reinforcing steel and buried metals can also be caused when the pH of the soil is too low or too high.

To determine the corrosion potential of the site soils on concrete, reinforcing steel, and buried metals, corrosivity analyses was completed on soil samples within the foundation embedment depths. Caltrans considers a site to be corrosive to foundation elements if any of the following conditions exist:

- Chloride concentration is greater than or equal to 500 ppm,
- Sulfate concentration is greater than or equal to 2,000 ppm,
- The pH is 5.5 or less,
- Resistivity is less than 1,500 Ohm-cm.

Boring No. / Depth	Minimum Resistivity (Ohm-Cm)	Chloride Content (ppm)	Sulfate Content (ppm)	pH
Boring B-1/ 8.5 ft				
Boring B-2/ 10.5 ft				

Based on the structure location and the results of the corrosion analyses, the site is considered [NON CORROSIVE OR CORROSIVE]. The corrosion test report is included in Appendix G.

9.0 SEISMIC RECOMMENDATIONS

9.1 GROUND MOTION INFORMATION

Deterministic and probabilistic acceleration response spectra (ARS) were generated using Caltrans ARS Online (Caltrans, 2013). The Caltrans ARS Online website describes how this web-based tool calculates spectra based on the criteria provided in Appendix B of Caltrans Seismic Design Criteria (Caltrans, 2013):

> *The deterministic spectrum is determined as the average of median response spectra calculated using the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations developed under the "Next Generation Attenuation" project coordinated through the PEER-Lifelines program. These equations are applied to all faults considered to be active in the last 750,000 years (late-Quaternary age) that are capable of producing a moment magnitude earthquake of 6.0 or greater. The probabilistic spectrum is obtained from the USGS (2008) National Hazard Map for 5% probability of exceedance in 50 years. Caltrans design spectrum is based on the larger of the deterministic and probabilistic spectral values. Both the deterministic and probabilistic spectra account for soil effects through incorporation of the parameter Vs30, the average shear wave velocity in the upper 30 meters of the soil profile.*

Shear wave velocities in the upper 30 meters of the soil profile (V_{s30}) were estimated using SPT blow count (ASTM D1586) correlations for cohesionless and cohesive soils adapted by Brandenberg et. al. (2010) (Caltrans, 2012). The shear wave velocities and site location were then input into the Caltrans ARS Online website to arrive at the controlling probabilistic scenario (CPS) and the ground motions summarized in the table below.

Fault Parameters						Site Parameters					
CPS	FID	Style	Dip (deg)	MM (max)	RRUP (km)	Vs30 (m/s)	PGA (g)	NFF	BAF	Z1.0 (m)	Z2.5 (km)
Great Valley 03a Dunnigan Hills	95	Rev	20 E	6.4	10.39	333	0.425	1	1.026	N/A	3.25

The seismic shear wave velocity and design ARS generated from the Caltrans ARS Online website are included in Appendix H.

9.2 SEISMIC HAZARDS

9.2.1 Liquefaction Potential

Liquefaction typically occurs in saturated near-surface soil layers consisting of poorly graded loose sands and gravels, and non-plastic silts (Kramer, 1996). The exploratory drilling operation revealed that the project site is generally underlain by alluvial deposits consisting of interbedded lean and fat clays and silts, and loose to very dense sands, gravels, and cobbles. Groundwater is located at the approximate elevation of Putah Creek. Results of the liquefaction analyses indicated the potential for seismic-induced distress to occur at the site as [LOW]. Liquefaction analyses results are included in Appendix H.

9.2.2 Surface Fault Rupture Potential

The site is not located within an Earthquake Fault Zone for active faults as defined by the State Geologist and the nearest mapped active fault (Great Valley 03a Dunnigan Hills) is located approximately 10 kilometers north of the site. Therefore, the potential for surface rupture due to primary faulting at the site is considered to be low.

9.2.3 Seismically-Induced Settlement

Seismically-induced ground shaking can cause vertical settlement of specific types of soils. Seismically related settlement generally results from the densification of loose sands and sandy silts due to vibrations or liquefaction. Our exploratory drilling operation revealed that the project site is generally underlain by layers of alluvial deposits consisting of interbedded lean and fat clays and silts, and loose to very dense sands, gravels, and cobbles. Due to the density and consistency of the soils encountered during our exploratory borings, the potential for seismically-induced settlement is [LOW].

9.2.4 Seismic Slope Instability

The creek banks at the site will be subject to seismic shaking during an earthquake. The inclinations of the creek banks range from 0.5H:1V to 2.5H:1V. The creek banks are currently statically stable. Minor raveling or shallow failures should be anticipated during or following a seismic event.

10.0 AS-BUILT FOUNDATION DATA

The as-built drawings and existing documentation for the bridge was obtained from Solano County. The as-built drawings (Solano County) indicate that the existing foundation supporting Pier 2 (South Pier) and Pier 3 (Center Pier) consist of standard timber piles. The existing foundation supporting Pier 4 (North Pier) consist of reinforced concrete piles. The pile lengths shown on the plans vary between Pier 2/Pier 3 and Pier 4. According to Kleinfelder's report dated April 28, 2006 (Kleinfelder, 2006), the timber standard piles and reinforced concrete piles have lengths of 40 feet and 15 feet, respectively. The pile capacities from Kleinfelder's analyses concluded that the standard timber piles have a capacity of 68 tons at Pier 2 and 53 tons at Pier 3; and the reinforced concrete piles at Pier 4 have a capacity of 44 tons.

The abutments are founded on spread footings approximately 2 feet thick. The spread footings for each abutment vary in depth below the paved surface. The footings at Abutment 1 (South Abutment) and Abutment 5 (North Abutment) are embedded approximately 6.5 feet and 11 feet, respectively, below the bridge approach road surface.

11.0 FOUNDATION RECOMMENDATIONS

11.1 SHALLOW FOUNDATIONS

11.1.1 Abutments

In the event that the Structural Designer (SD) determines that the existing shallow foundations at the abutments have the required load capacity and behave in a rigid manner, then a reduced combination of passive and frictional resistance can be used to design against sliding. When both frictional and passive resistance is used, the designer should only account for a portion of the full passive resistance. The applicable method for combining friction and passive resistance against sliding is provided in Section 10.6 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2012).

Based on materials encountered in the exploratory borings from a previous investigation completed by Kleinfelder, the existing footings are placed on silt to sandy silt material. An effective friction angle of 28 degrees should be used and the friction coefficient should be calculated as the tangent of the friction angle. The effective friction angle was determined based on materials underlying the existing footings at the abutments using SPT correlations provided in the 2014 Caltrans Geotechnical Manual (Caltrans, 2014). Strength contributions from cohesion should be neglected.

An allowable passive equivalent fluid pressure (EFP) of 260 psf/ft can be used for passive resistance from the abutment back-walls. This value represents the full passive resistance and should be reduced by 50 percent when used in combination with frictional resistance. The passive EFP is based on an average unit weight of the soil column retained by the abutment back-walls. It is assumed that the backfill is level and passive resistance from the slope face side of the footings is neglected. When accounting for passive resistance in the transverse direction, passive resistance from the upper two feet should be neglected.

11.1.2 Piers

Shallow foundations would require significant excavation and backfill to construct due to the relatively deep embedment at the pier locations, where the footing would need to be constructed well below the creek bed. In addition, the relatively large excavation area would increase the potential for conflict with existing improvements. For these reasons, shallow foundations are not recommended for piers.

11.2 DEEP FOUNDATIONS

It is recommended that the bridge abutments and piers be supported on a deep foundation system in order to minimize the construction footprint and limit the quantity of excavations, and reduce the potential for conflict with existing utilities and improvements.

Either cast-in-drilled hole (CIDH) concrete piles or driven piles could potentially be used. However driven piles may not be as economical compared to CIDH concrete piles due to the high mobilization cost relative to the number of piles needed and the large construction footprint required to drive piles.

The allowable pile bearing capacity for 60 inch and 84 inch diameter CIDH piles were analyzed. End bearing was neglected in the analyses. The abutment piles and pier piles were analyzed separately since the top of pile elevation and slope geometry were significantly different between the two.

11.2.1 Abutments

Support	Pile Type	Cut-off Elev (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
			Total	Permanent				
Abut 1	60" CIDH							
Abut 5	60" CIDH							

11.2.2 Piers

Support	Pile Type	Cut-off Elev (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
			Total	Permanent				
Pier 2	84" CIDH							
Pier 3	84" CIDH							
Pier 4	84" CIDH							

11.3 APPROACH FILL EARTHWORK

Minor earthwork is expected at the location of the bridge abutments. Clearing and grubbing of the vegetation, pavement, cobbles, boulders, etc. and all subsequent earthwork shall conform to Section 16 "Clearing and Grubbing", and Section 19, "Earthwork", of the Caltrans Standard Specifications, 2010 edition (Caltrans, 2010). After clearing and grubbing, any exposed subgrade soils, on which the abutments will be formed, should be scarified to a minimum depth of 12 inches, moisture conditioned, and compacted to a firm and level base. The fill should be keyed and benched into the existing slope.

11.4 ROADWAY REALIGNMENT

[SECTION TO BE COMPLETED IN FINAL DRAFT]

12.0 CONSTRUCTION CONSIDERATIONS

The following items should be considered during construction:

- Groundwater will be encountered during the excavation of the drilled shafts. Temporary casing of the drilled shafts will be required.

- Loose sands, gravels, and cobbles susceptible to caving were encountered in all borings during subsurface exploration. These granular materials will cave into the drilled shafts during construction of CIDH piles and the contractor should be prepared to install temporary casing on-site before drilling of CIDH piles. The contractor should evaluate the need for rotator or oscillator casing.

- Proper tremie embedment should be maintained during pile concrete placement.
- Excavations should be sloped or shored in conformance with OSHA requirements for Type C Soil.

13.0 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the planned construction, and the results of the subsurface exploration and testing, combined with interpolation of the subsurface conditions between boring locations. This information notwithstanding, the nature and extent of subsurface variations between borings may not become evident until construction. It is recommended that Cal Engineering & Geology be retained to observe the pile drilling and earthwork operations to confirm the subsurface conditions between the exploratory borings are as estimated. If variations are encountered during construction, Cal Engineering & Geology should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

This report was prepared based on preliminary design information which is subject to change during the design process. At approximately the 90 percent design level, Cal Engineering & Geology should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Cal Engineering & Geology should be provided the opportunity to review those portions of the plans and special provisions that pertain to bridge foundation and earthwork and related operations and items of work to determine whether they are consistent with the recommendations of this report. It is Quincy Engineering's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes. In the event Cal Engineering & Geology is not retained for review, we assume no liability for the misrepresentation of our conclusions and recommendations.

Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

This report presents the results of a geotechnical subsurface exploration only and should not be construed as an environmental audit or study. The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

14.0 REFERENCES

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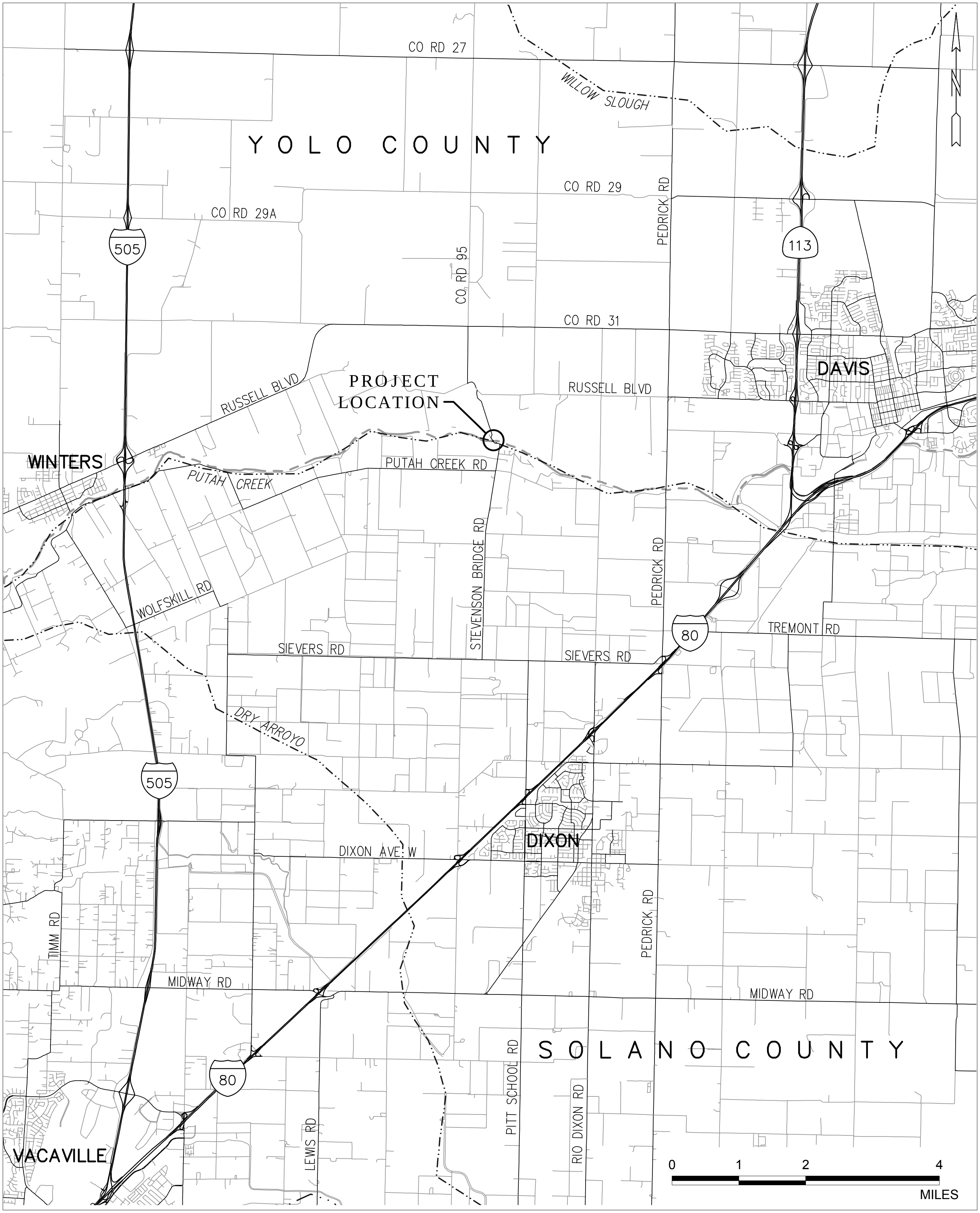
Appendix A. Vicinity Map



CE&G
CAL ENGINEERING & GEOLOGY

1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

STEVENSON BRIDGE OVER PUTAH CREEK
STEVENSON BRIDGE ROAD
SOLANO COUNTY AND YOLO COUNTY, CALIFORNIA

VICINITY MAP

160600	OCTOBER 2016	APPENDIX A

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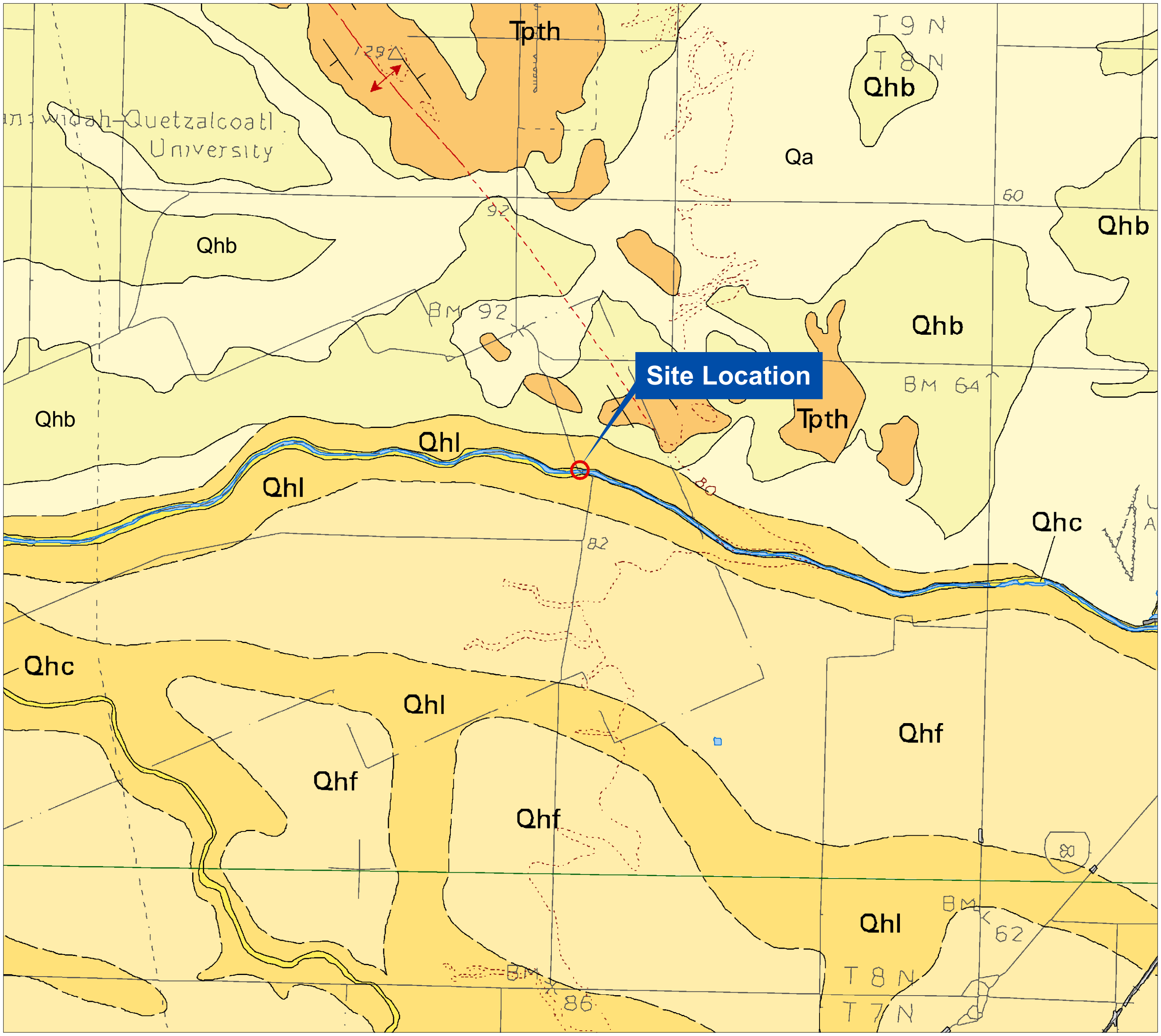
Appendix B. Regional Geology Map



BASEMAP REFERENCE

1. GEOLOGIC MAP AND MAP DATABASE OF NORTHEASTERN SAN FRANCISCO BAY REGION, CA, BY GRAYMER AND OTHERS, 2002

0 0.5 1 2
MILES

N

MAP UNIT DESCRIPTION

SURFICIAL DEPOSITS

Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhc	STREAM CHANNEL DEPOSITS (HOLOCENE)
Qhl	NATURAL LEVEE DEPOSITS (HOLOCENE)
Qhb	BASIN DEPOSITS (HOLOCENE)
Qa	ALLUVIUM (HOLOCENE AND LATE PLEISTOCENE)

VACAVILLE ASSEMBLAGE

Tpth	TEHAMA FORMATION (PLIOCENE)

M:\2016\160600 Quincy - Stevenson Rd. Bridge\GIS\160600_Fig3_Geology.mxd; 10/28/2016; kdrozynska

CE&G
CAL ENGINEERING & GEOLOGY

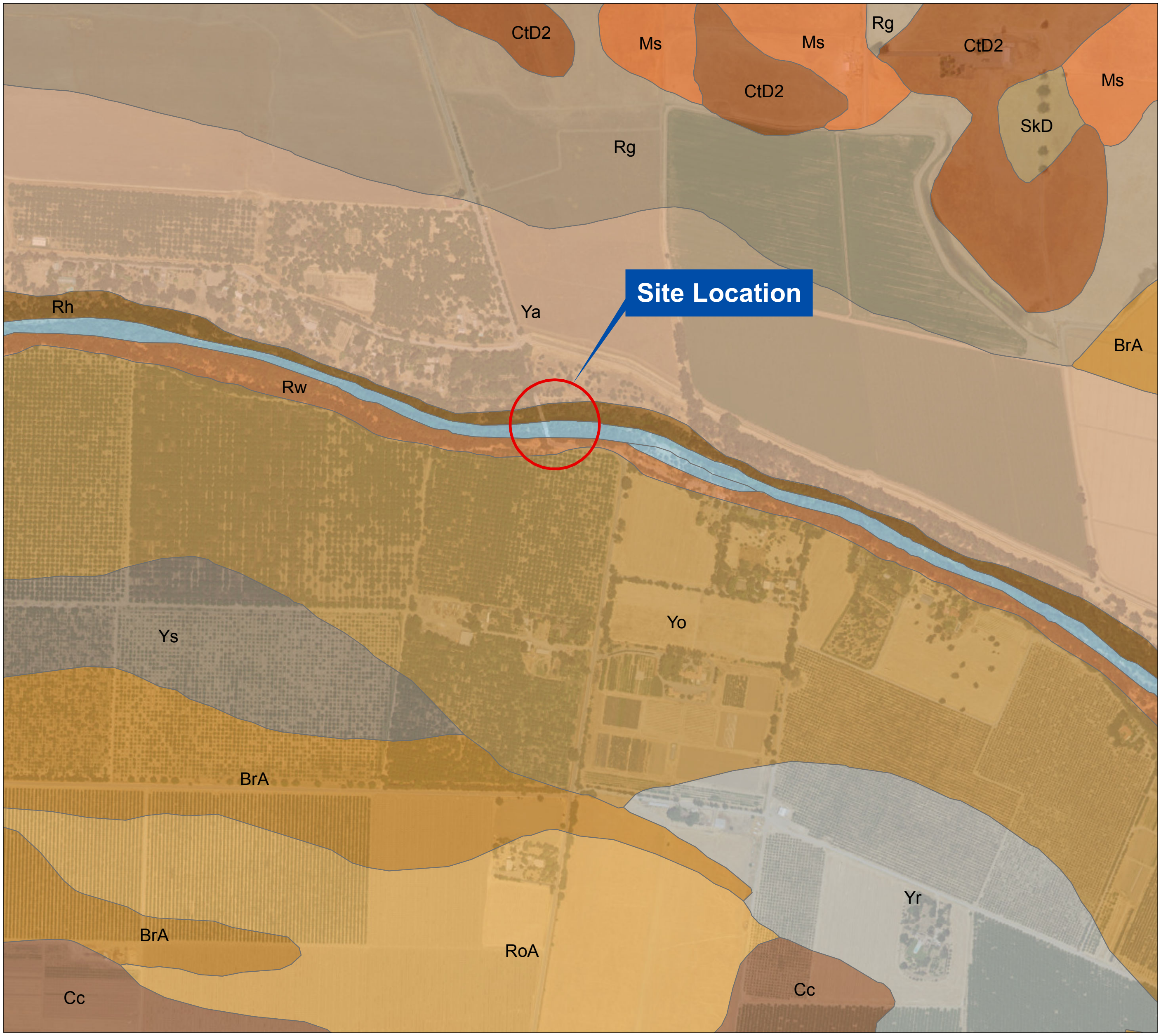
1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

STEVENSON BRIDGE OVER PUTAH CREEK
STEVENSON BRIDGE ROAD
SOLANO COUNTY AND YOLO COUNTY, CALIFORNIA

REGIONAL GEOLOGY MAP

160600	OCTOBER 2016	APPENDIX B

Appendix C. NRCS Soil Map



BASEMAP REFERENCE

1. SOIL SURVEY STAFF, NATURAL RESOURCES CONSERVATION SERVICE, UNITED STATES DEPARTMENT OF AGRICULTURE. WEB SOIL SURVEY. AVAILABLE ONLINE, ACCESSED 30 JUNE 2016.

0 500 1,000 2,000 FEET

N

MAP UNIT DESCRIPTION

SOLANO COUNTY, CA (CA095)

BrA	BRENTWOOD CLAY LOAM, 0 TO 2 PERCENT SLOPES
Cc	CAPAY CLAY
RoA	RINCON CLAY LOAM, 0 TO 2 PERCENT SLOPE
Rw	RIVERWASH
Yo	YOLO LOAM, 0 TO 4 PERCENT SLOPES, MLRA 17
Yr	YOLO LOAM, CLAY SUBSTRATUM
Ys	YOLO SILTY CLAY LOAM, 0 TO 2 PERCENT SLOPES, MLRA 17

YOLO COUNTY, CA (CA113)

BrA	BRENTWOOD CLAY LOAM, 0 TO 2 PERCENT SLOPES
CtD2	CORNING GRAVELLY LOAM, 2 TO 15 PERCENT SLOPES, ERODED
Ms	MYERS CLAY
Rg	RINCON SILTY CLAY LOAM
Rh	RIVERWASH
SkD	SEHORN CLAY, 2 TO 15 PERCENT SLOPES
Ya	YOLO SILT LOAM, 0 TO 2 PERCENT SLOPES, MLRA 17

CE&G
CAL ENGINEERING & GEOLOGY

1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

STEVENSON BRIDGE OVER PUTAH CREEK
STEVENSON BRIDGE ROAD
SOLANO COUNTY AND YOLO COUNTY, CALIFORNIA

NRCS SOIL MAP

160600	OCTOBER 2016	APPENDIX C

Appendix D. CE&G Boring Logs

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
D. Burger	**9-12-16**	**9-14-16**	**38.5 ft / -121.9 ft**	**B-1**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Woodward Drilling				**100.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Rotary Wash			**Mobile B57**	**3-7/8**
SAMPLER TYPE(S) AND SIZE(S) (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Std Cal (2.5"), SPT (1.4")			**140 lb / 30 in autotrip**	**76.5%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS / DURING DRILLING: **Not determined** / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat cement				**121.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	SILT (ML); light brown; dry; few fine SAND ; medium stiff consistency; (Alluvium).								
98.00	2	Few sub-rounded to sub-angular gravel up to 1 in. in cuttings.								
96.00	4	Increased moisture at 4 ft.								
	4.5	SILT (ML); brown variegated with red brown and gray; moist; few fine SAND ; mostly low plasticity, low dry strength, rapid dilatancy FINES ; hard consistency; few sand to little in lenses.	3, 4, 3	7	78		25	101		
94.00	6		3, 3, 5	8	100					PA
	6.5	SILTY SAND (SM); silty sand lense less than 2 in.								
	7	Elastic SILT (MH); elastic silt and lean clay with sand lenses less than 4 in. thick.								
92.00	8		5, 5, 8	13	94		26	102		
	9.5	SILT with SAND (ML); brown; wet; some fine SAND ; mostly low plasticity, rapid dilatancy FINES ; stiff consistency; gradational contact.	3, 5, 6	11	89					PA
90.00	10	Elastic SILT (MH); brown; moist; few SAND ; mostly low to medium plasticity, slow dilatancy FINES ; stiff consistency; lense of light gray, lean clay with sand lens at 10.5 ft.								
86.00	13.5	Lean CLAY (CL); strong brown to brown with variegated light gray; moist; trace SAND ; mostly medium plasticity, slow to none dilatancy FINES ; hard consistency; charcoal at 15 ft., thin lens of elastic silt to elastic silt with sand at 15-15.5 ft.	8, 19, 22	41	78		20	110		
	15		5, 7, 9	16	100					PA, PI
82.00	18.5	Elastic SILT (MH); brown; moist; few SAND ; mostly low to medium plasticity, slow dilatancy FINES ; medium stiff consistency; gradational contact to ML at 19.75.	3, 4, 6	10	100					
80.00	20	SILT (ML); brown; wet; few SAND ; mostly low plasticity, low dry strength, rapid dilatancy FINES ; soft consistency; thin lens of elastic silt between 20.5 to 21.0 ft.	2, 4, 5	9	100					PA
76.00	23.5	SANDY SILT (ML); brown; wet; some fine SAND ; low to none plasticity, rapid dilatancy FINES ; stiff consistency.	5, 9, 9	18	61		28	100		
	25	Poorly graded SAND with SILT (SP-SM); medium		15	100					

(continued)

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1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

REPORT TITLE				HOLE ID
BORING RECORD				**B-1**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D.**		DATE	SHEET **1 of 5**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25	dense; brown gray; wet; fine SAND ; few FINES. Elastic SILT (MH); olive brown; Iron stains.			3 5 10	15	100							
74.00	26	Well-graded GRAVEL (GW); fine GRAVEL in the bottom of sample. Driller indicated fluid loss and coarse sand and sub-angular GRAVEL up to 1/2" in cuttings, thick and drilling fluid.												
	27													
72.00	28				28 50		82		9					
	29	Well-graded GRAVEL with lenses of well graded gravel with sand very dense; dark gray; wet; subangular to rounded up to 2" GRAVEL ; little fine to coarse SAND ; weak cementation; consisting of chert, greenstone. graywacke.			9 30 22	52	67							
70.00	30	Well-graded SAND with GRAVEL (SW); very dense; dark brown to gray; wet; few rounded to rounded up to 1/2" GRAVEL ; trace FINES ; weak cementation.												
	31													
68.00	32	Well-graded GRAVEL (GW); driller indicate large gravels and cobbles with loss of fluid - mixed even thicker fluid. Cobbles at 32.5 ft. Medium dense; gray; greater than 1.25", rounded GRAVEL ; well-graded GRAVEL with cobbles at bottom of sample.. Driller indicated "Out of gravel" change at 36 ft.												
	33				4 14 8	22	33							
66.00	34													
	35													
64.00	36	Lean CLAY (CL).												
	37	Well-graded GRAVEL (GW); thin gravel lens at 37 ft. Less than 6 in. as indicated by sound and cuttings. Approx. 8 in. of caving.												
62.00	38	Well-graded GRAVEL (GW). Driller indicated sand and gravel (coarse sand and fine gravels in cutting). Loose as indicated by driller.												
	39													
60.00	40													
	41	GRAVEL softer at 41 ft, more sand, less gravel. Grades.												
58.00	42	SANDY lean CLAY (CL); brown; fine to coarse SAND ; in cuttings.												
	43													
56.00	44													
	45	8 ft. of caving at 45 ft, sample interval, continued drilling with sample.												
54.00	46	Well-graded SAND (SW); few fine up to 1/4" GRAVEL ; medium to coarse SAND ; 8 ft of caving at sample interval, large gravel in hole. Elected to case hole from 0-48.5.												
	47													
52.00	48	Lean CLAY (CL); olive brown; moist; trace fine SAND ; medium plasticity FINES ; very stiff consistency; sharp contact with lens of well-graded SAND with GRAVEL.			15 50		75		14	124				
	49	Well-graded SAND with GRAVEL (SW); very dense; dark brown gray; wet; some fine up to 3/4", rounded GRAVEL ; trace medium to very coarse SAND ; trace FINES.			14 23 23	46	78							
50.00	50													
	51	Lean CLAY (CL); olive brown grades to light brown; moist; about 1/2", rounded GRAVEL ; few very fine SAND ; mostly medium plasticity, slow to none dilatancy FINES ; very stiff consistency; isolated rounded GRAVEL at the bottom of sample.												
48.00	52													
	53													
46.00	54													
	55													

(continued)

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1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

REPORT TITLE **BORING RECORD**				HOLE ID **B-1**	
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**	
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**					
BRIDGE NUMBER	PREPARED BY **K. D.**			DATE	SHEET **2 of 5**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
44.00	55–56.5	Driller indicated stiff to very stiff CLAY. 30 minutes to drill 51-56.5 ft.. Lean CLAY (CL) *(continued)*.								
	56.5–57.5	Well-graded SAND (SW); driller indicated SAND at 56.5. Thin lens (1 ft thick).								
42.00	57.5–60.5	SANDY SILT (ML); olive brown; moist; some very fine to medium SAND ; mostly low plasticity, rapid dilatancy FINES ; hard consistency; lenses of SILT with SAND.	21 21 23	44	61					
40.00			6 8 13	21	0					PA
38.00	60.5–65	SILT (ML); olive brown; moist; trace SAND ; low plasticity to none, rapid dilatancy FINES ; very stiff consistency; grades to SILT.								
34.00	65–68	Well-graded SAND with SILT and GRAVEL (SW-SM); Driller indicated SAND and GRAVEL between 65-68 ft.								
32.00	68–77	Well-graded SAND with GRAVEL (SW); very dense; dark brown gray; wet; rounded to subrounded GRAVEL ; medium to very coarse SAND ; few FINES ; weak cementation; rounded to subrounded GRAVEL with few angular gravel up to 3/4" consisting of quartz, chert, graywacke.	25 54		83					
22.00	77–80	SILT with SAND (ML); brown; moist; few very fine to fine SAND ; nonplastic to low plasticity, low dry strength, rapid dilatancy FINES ; hard consistency; decrease SAND in cuttings at 77 ft.	12 23 36	59	78		21	107		
20.00	80–85	Elastic SILT (MH); brown; moist; trace SAND ; mostly medium plasticity, slow dilatancy FINES ; stiff consistency.	7 10 12	22	89					PA
16.00	83–85	Used 3-7/8" clay bit. 15 minutes to drill 83-88 ft. Driller indicated clay from 81-88 ft.								

(continued)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85	Elastic SILT (MH) *(continued)*.												
14.00	86	Assumed contact based on driller indicating slightly sandier and asier drilling conditions.												
	87	SILT with SAND (ML); brown; moist; little very fine to fine SAND ; low plasticity, rapid dilatancy FINES ; very stiff consistency; trace subangular sandstone rock fragments less than 5% in the bottom 6" of sample.												
12.00	88				7 12 19	31	89							
	89	Some SAND.												
10.00	90													
	91	Thin sand lens less than 6" thick at about 91 ft as incredibly dark.												
8.00	92	About 35 minutes to drill from 89.5 to 93 ft. Driller indicated stiff clays.												
	93													
6.00	94	About 20 minutes to drill from 93-98 ft (~ 4 min/ft). Clayey / silty throughout as indicated by driller.												
	95													
4.00	96													
	97	Sandy silt on the end of drill bit.												
2.00	98	SILTY SAND (SM); dense; brown; moist; very fine to fine SAND ; little FINES ; weak cementation; Gradational contact to.			17 35 36	71	78							
	99	Poorly graded SAND (SP); very dense; gray brown; moist; fine to medium SAND ; few FINES ; gradational contact.							22	103				
0.00	100				7 9 15	24	100							
	101	Elastic SILT (MH); brown; moist; trace very fine SAND ; low to medium plasticity, rapid to slow dilatancy FINES ; stiff consistency; trace to few very fine sand in lenses less than 1/2 in. Isolated round sandstone rock fragments less than 1/4 in.												
-2.00	102													
	103													
-4.00	104													
	105	Approx. 1 hour to drill 101 to 109 ft. Clayey throughout with a few thin sandy lenses less than 6 in. as indicated by driller.												
-6.00	106													
	107													
-8.00	108	Lean CLAY (CL); Assumed contact. Thin lens of lean CLAY in upper 4 in of sample.												
	109	Elastic SILT (MH); brown; moist; few very fine SAND ; medium plasticity, slow dilatancy FINES ; very stiff consistency.			12 17 22	39	100							
-10.00	110	SILT with SAND (ML); few very fine SAND ; low plasticity, rapid dilatancy FINES.												
	111	Lean CLAY (CL); Driller indicated stiffer drilling and clayey at about 111.5 ft. Drilling from 110.5 to 118 about 45 minutes. Clayey throughout.												
-12.00	112													
	113													
-14.00	114													
	115													

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		Lean CLAY (CL) *(continued)*.												
-16.00	116														
	117														
-18.00	118		Lean CLAY (CL); brown; moist; trace very fine SAND ; mostly medium plasticity, none to slow dilatancy FINES ; hard consistency.			21 34 48	82	83							
	119														
-20.00	120					10 20 24	44								
	121		Lean CLAY (CL); brown to olive brown; moist; trace SAND ; medium plasticity FINES ; hard consistency; with lens of 5-10% SAND.												
-22.00	122		Bottom of borehole at 121.0 ft bgs												
	123														
-24.00	124		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	125														
-26.00	126														
	127														
-28.00	128														
	129														
-30.00	130														
	131														
-32.00	132														
	133														
-34.00	134														
	135														
-36.00	136														
	137														
-38.00	138														
	139														
-40.00	140														
	141														
-42.00	142														
	143														
-44.00	144														
	145														

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
D. Burger	**9-21-16**	**9-22-16**	**38.5 ft / -121.9 ft**	**B-2**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Woodward Drilling		**104.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Mobile B57**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Std Cal (2.5"), Mod Cal (2"), SPT (1.4")	**140 lb / 30 in autotrip**	**76.5%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **Not determined**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Neat cement		**129.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	Moisture Content (%)	Remarks
	0	SILT (ML); SILT and 2 ft. minus rip-rap. Hollow stem (6") auger used from 0-10 ft. About 1 hour to advance through soil/rip-rap layers to sample depth of 5 ft.					
102.00	2						
	3	Water at 3 ft at time of drilling. Driller indicated out of rip-rap zone at 4 ft.					
100.00	4	Well-graded SAND with CLAY and GRAVEL (SW-SC); very dense; gray brown; wet; little fine up to 1/2", subrounded GRAVEL ; fine to very coarse SAND ; little FINES.					
	5		9, 41, 31	72	50	9	
	6.5		7, 3, 4	7	50		PA
	7	Well-graded GRAVEL with SILT and SAND (GW-GM); loose; gray brown; wet; fine up to 3/4 in, rounded GRAVEL ; some very fine to very coarse SAND ; few FINES.					
94.00	10		2, 7, 15	22	78	25	
	10.5	Lean CLAY (CL); brown; moist; trace SAND ; medium plasticity, none to slow dilatancy, low toughness FINES ; stiff consistency; switched to rotary wash at 13 ft using 6" HSA as conductor casing.					
92.00	11.5		4, 6, 9	15	67		
	13	Elastic SILT (MH); brown; moist; trace SAND ; medium grades to low plasticity, slow dilatancy FINES ; stiff consistency.	4, 6, 7	13	89	32	
	14.5	Variegated light blue, gray in brown elastic silt matrix; Thin lean clay lenses less than 2".	2, 3, 3	6	61		PA
	17	SILT with SAND (ML); brown; wet; little very fine SAND ; mostly low plasticity, low dry strength, rapid dilatancy FINES ; stiff consistency; assumed contact.					
	18		4, 6, 7	13	78	32	
84.00	19.5		3, 3, 5	8	100		PA
	21.5	SILT with SAND (ML); brown; wet; little very fine to fine SAND ; low plasticity, rapid dilatancy FINES ; medium stiff consistency.					
82.00	22	Well-graded GRAVEL with SAND (GW); dense; brown gray; wet; mostly fine, less than 1/2" GRAVEL ; some fine to very coarse SAND ; trace FINES ; Driller indicated gravel at 22 ft. Rounded to subrounded gravel up to 1.5". Consisting of quartz and graywacke.					
	23		10, 19, 25	44	67	10	
80.00	24.5			21	44		
	25						

(continued)

CE&G
Cal Engineering & Geology
1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

REPORT TITLE				HOLE ID
BORING RECORD				**B-2**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D**		DATE	SHEET **1 of 5**

Depth (ft)	Description	Blows per 6 in.	Blows per foot	Recovery (%)	Moisture Content (%)	Remarks
25	Well-graded GRAVEL with SAND (GW) *(continued)*.	3 5 16	21	44		
25.5	Well-graded SAND with GRAVEL (SW); medium dense; brown gray; wet; mostly fine up to 1/2", rounded GRAVEL ; fine to very coarse SAND ; with isolated trace rounded gravel up to 1". Loss of circulation at 27 ft, mixed thicker drilling fluid.					
28	Well-graded GRAVEL with SAND (GW/GW); dense; brown gray; wet; coarse up to 2.5 in. GRAVEL ; some medium to very coarse SAND. Cobbles larger than 2.5 ft at 29 ft.	7 24 31	55	67		
29.5		4 12 20	32	39		
31	About 60% well-graded coarse sand at bottom of sample near 31 ft with about 15% fines.					
31.5	SILT with SAND (ML); brown; wet; 15% very fine to fine SAND ; mostly low plasticity, rapid dilatancy FINES ; stiff consistency; driller indicated CLAYEY at 31.5 ft GRAVELS advance while drilling sample creating disturbed sample with limited recovery betweeen 33-34.5 ft. Thin lenses less than 4 in. of sandy SILT with about 40% very fine to fine SAND.					
33		8 30 34	64	50		
34.5		9 12 16	28	78		
37.5	Poorly graded SAND (SP); dense; dark brown gray; wet; trace fine up to 1/4 in., isolated rounded GRAVEL ; fine to medium SAND ; trace FINES ; dense consistency; approx. contact at 37.5 based on drilling resistance.					
38		23 34 36	70	78		
39.5		3 2 10	12	33		PA
40	Poorly graded SAND (SP); medium dense; dark gray; wet; fine to medium SAND ; trace FINES.					
43	Poorly graded SAND with GRAVEL (SP); very dense; dark gray; wet; little fine up to 3/8", subrounded GRAVEL ; medium SAND ; trace FINES.	31 50.5		82	14	
44		6 27 27	54			
46	Lean CLAY (CL); Driller indicated stiffer drilling resistance and clayey at 46 ft.					
48	Lean CLAY (CL/CL); olive brown variegated with light blue gray veins; moist; trace very fine SAND ; mostly low plasticity, slow dilatancy FINES ; hard consistency; some silt.	12 27 45	72	50		
49.5		10 16 19	35	61		
51	Time to drill 51-58 ft is 55 min.					
52	Clayey while drilling, possible thin SAND lenses a few inches thick at various depths based on drilling resistance, easing up briefly.					

Other column headers (no entries on this page): Sample Location, Sample Number, RQD (%), Dry Unit Weight (pcf), Shear Strength (tsf), Drilling Method, Casing Depth.

ELEVATION (ft)	DEPTH (ft)
78.00	26
76.00	28
74.00	30
72.00	32
70.00	34
68.00	36
66.00	38
64.00	40
62.00	42
60.00	44
58.00	46
56.00	48
54.00	50
52.00	52
50.00	54

(continued)

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1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

REPORT TITLE				HOLE ID
BORING RECORD				**B-2**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D**		DATE	SHEET **2 of 5**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		Lean CLAY (CL) *(continued)*.												
48.00	56														
	57														
46.00	58		Driller indicated clay at 58 ft with clay on drill bit.												
			Poorly graded SAND (SP); dense; gray brown; wet; fine to medium SAND ; trace FINES ; isolated rounded to subrounded gravel at top of sample near 58.5 ft.			24 36 45	81	89							
	59									19					
44.00	60		No recovery of sample. Assumed SP as above.			2 5 19	24								
	61														
42.00	62		Drill time 20 minutes for 61-68 ft.												
	63		Driller indicated sandy and easy drilling conditions.												
40.00	64		Thin gravel lens (less than 4") at 64 ft.												
	65														
38.00	66		Thin gravel lens (6") at 66 ft.												
	67		Thin gravel lens (3") at 67 ft.												
36.00	68		Lean CLAY (CL); Driller noted change and stiffer drilling conditions at 67.5 ft. Lean CLAY (CL); brown; moist; trace SAND ; medium plasticity, none to slow dilatancy, medium toughness FINES ; hard consistency.			11 11 14	25	83							
	69														
34.00	70		Drilling time 69.5-78 ft is 20 minutes.												
	71														
32.00	72		Driller noted varible drilling resistance indicating interbedded clays and sands but primarily clay.												
	73														
30.00	74														
	75														
28.00	76		SANDY SILT (ML); Approx. contact based on drilling resistance and increased drilling rate.												
	77														
26.00	78					15 30 45	75	78							
	79		SILT grades to SANDY SILT brown; moist; mostly low plasticity, rapid dilatancy FINES ; hard consistency; sand grades from 5-10% to 25-35% very fine with depth in sample.												
24.00	80		Poorly graded SAND with SILT (SP-SM); dense; gray brown; wet; very fine to fine SAND ; few FINES ; 10% fines grades to trace fines.			3 12 22	34	100							
	81		SILT (ML); 2 in. silt lens at 80.75 ft, below silt lens medium to coarse sand, gray, trace to none fines.												
22.00	82		Lean CLAY (CL); stiff consistency; driller indicated stiff clay at 82 ft. Drilling 81-88 ft in 50 minutes.												
	83														
20.00	84		Assumed stiff clay based on slow drill rate.												
	85														

(continued)

CE&G Cal Engineering & Geology
1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

REPORT TITLE				HOLE ID
BORING RECORD				**B-2**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D**	DATE		SHEET **3 of 5**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85	Lean CLAY (CL) *(continued)*.										
18.00	86	Drilling eased up slightly at 86.5 ft.										
	87	Elastic SILT (MH).										
16.00	88	Elastic SILT (MH); brown; moist; trace SAND ; mostly medium plasticity, slow dilatancy FINES ; stiff consistency. Silt lense with low plasticity at bottom 2" of sample.	5 9 10	19	100							
	89											
14.00	90											
	91	Drilling 84.5-98 ft in 45 min.										
12.00	92	Lean CLAY (CL); Stiff drilling likely clay or elastic silt.										
	93											
10.00	94	Lean CLAY (CL); Assumed clay based on drilling rate.										
	95											
8.00	96											
	97	SILTY SAND (SM); dense; brown; wet; very fine to fine SAND ; some FINES ; Drilling eased up at 97 ft. Sand grades to SP.										
6.00	98		15 37 45	82	78		20					
	99	Poorly graded SAND (SP); very dense; dark brown gray; wet; fine to medium SAND ; trace FINES.										
4.00	100	Poorly graded SAND with thin SANDY SILT lenses dense; dark brown gray with brown lenses; mostly fine to medium SAND ; 10% FINES ; Coarse SAND lenses near 101 ft. SILT lenses up to 2 in. thick but most are about 1/2 in. thick.	11 18 34	52	83							
	101											
2.00	102	Drill time for 101-108 ft is 40 minutes.										
	103	Drilling resist increased at about 103 ft indicating CLAY.										
0.00	104	Elastic SILT (MH).										
	105											
-2.00	106											
	107											
-4.00	108	Elastic SILT (MH); olive brown; wet; trace SAND ; medium plasticity, slow dilatancy FINES ; medium stiff consistency; thin CLAY lenses, less than 2 in. thick.	7 9 12	21								
	109											
-6.00	110	Drill time between 109.5-118 ft is 66 minutes.										
	111											
-8.00	112											
	113											
-10.00	114	Lean CLAY (CL/CL); Very hard drilling resistance betweeen 114-118 ft.										
	115											

(continued)

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REPORT TITLE				HOLE ID
BORING RECORD				**B-2**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D**		DATE	SHEET **4 of 5**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		Lean CLAY (CL/CL) *(continued)*.												
-12.00	116														
	117														
-14.00	118		Lean CLAY (CL/CL); olive brown variegated with light gray; moist; trace SAND ; medium plasticity, slow to none dilatancy FINES ; hard consistency; discontinous lenses.			23 38 50.5	88.5	94							
	119														
-16.00	120		Lean CLAY (CL/CL); drilling time 121-128 ft is 55 min.			15 23 25	48								
	121														
-18.00	122														
	123		Hard drilling, resist throughout run with possible very thin sandy beds.												
-20.00	124														
	125														
-22.00	126														
	127														
-24.00	128		Lean CLAY (CL/CL); brown gray; wet; trace SAND ; medium plasticity, slow to none dilatancy FINES ; very stiff consistency.			12 14 15	29	100							
	129														
-26.00	130		Bottom of borehole at 129.5 ft bgs												
	131														
-28.00	132		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	133														
-30.00	134														
	135														
-32.00	136														
	137														
-34.00	138														
	139														
-36.00	140														
	141														
-38.00	142														
	143														
-40.00	144														
	145														

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REPORT TITLE				HOLE ID	
BORING RECORD				B-2	
DIST. 04	COUNTY SOL	ROUTE	POSTMILE	PROJECT ID 160600	
PROJECT OR BRIDGE NAME Quincy - Stevenson Rd. Bridge					
BRIDGE NUMBER	PREPARED BY K. D			DATE	SHEET 5 of 5

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
E. Zane	**10-18-16**	**10-20-16**	**38.5 ft / -121.9 ft**	**B-3**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Woodward Drilling		**82.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Mobile B57**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Std Cal (2.5"), SPT (1.4")	**140 lb / 30 in autotrip**	**76.5%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **Not determined** — AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Neat cement		**139.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		SILT (ML); dry; firm.												
	1														
80.00	2														
	3														
78.00	4														
	5		SILT with SAND (ML); light brown; dry; little fine SAND ; low plasticity, low dry strength, rapid dilatancy FINES ; weak cementation; hard consistency.		3-1 3-2	5 6 7	13	56							
76.00	6														
	7				3-3	5 4 9	13	28							
74.00	8														
	9														
72.00	10														
	11				3-4 3-5	5 6 7	13	72							
70.00	12		Well-graded SAND with SILT (SW-SM); medium dense; brown; moist; fine to medium, subrounded to subangular SAND.		3-6	5 6 5	11	33							
	13														
68.00	14		SILT with SAND (ML); brown; moist; little fine SAND ; low plasticity, low dry strength, slow dilatancy, low toughness FINES ; soft consistency.												
	15		SILTY SAND (SM); medium dense; light brown; moist; fine to medium, subrounded to subangular SAND ; 30% SILT.												
66.00	16														
	17				3-7 3-8	5 8 11	19	78							
64.00	18				3-9	5 6 7	13	39							
	19		SILTY SAND with GRAVEL (SM); medium dense; brown; moist; 5% subrounded, flat and elongated GRAVEL ; medium, subangular SAND ; 30% SILT.												
62.00	20														
	21		Well-graded GRAVEL with SAND (GW); medium dense; brown; wet; fine, subrounded to subangular GRAVEL.												
60.00	22		Well-graded SAND with SILT (SW-SM); medium dense; brown; moist; fine to medium, subrounded to subangular SAND ; 15% SILT.		3-10 3-11	9 12 12	24	44							
	23				3-12	9 11 10	21	39							
58.00	24		SILTY SAND with GRAVEL (SM); medium dense; brown; moist; few fine to coarse, subangular GRAVEL ; 15% SILT.												
	25														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		SILTY SAND with GRAVEL (SM) *(continued)*. Driller noted lost all fluid at 24 ft.												
56.00	26		Well-graded GRAVEL (GW); dense; greenish gray; wet; fine to coarse, rounded to subangular GRAVEL.		3-13	9 14 17	31	78							
	27		Well-graded SAND with SILT (SW-SM); olive grad; dry; fine, subrounded to subangular SAND ; 10% SILT.												
54.00	28		SANDY lean CLAY (CL); firm, yellowish orange; dry; some fine SAND ; medium plasticity, low dry strength, no dilatancy, low toughness FINES.		3-14	13 13 19	32	44							
	29		CLAYEY SAND (SC); dense; yellowish orange; moist; fine SAND ; weak cementation; 40% LEAN CLAY.												
52.00	30		Lean CLAY (CL); mottled greenish gray, yellowish orange; moist; medium plasticity, medium dry strength, no dilatancy, medium toughness FINES ; very stiff consistency.		3-15 3-16	12 19 25	44	78							
	31				3-17	11 17 18	35	39							
50.00	32		SILT with SAND and CLAY mottled greenish gray to yellowish orange; fine SAND ; low plasticity, medium dry strength, rapid dilatancy, low toughness FINES ; hard consistency.												
	33														
48.00	34														
	35				3-18 3-19	9 21 29	50								
46.00	36		SILTY SAND (SM); medium dense; light brown; moist; fine SAND ; 40% SILT.		3-20	9 13 15	28								
	37		Lean CLAY with SAND (CL); light brown; moist; some fine SAND ; medium plasticity FINES ; hard consistency; little angular coarse rock.												
44.00	38		SILT with SAND (ML); mottled light brown, greenish gray; moist; fine SAND ; medium plasticity, medium dry strength, slow dilatancy, low toughness FINES ; medium stiff consistency.												
	39														
42.00	40		Lean CLAY (CL).												
	41		Lean CLAY (CL); light brown; moist; medium plasticity, high dry strength, no dilatancy, medium toughness FINES ; very stiff consistency.		3-21 3-22	9 16 23	39	89							
40.00	42														
	43		Same.		3-23	9 15 18	33	100							
38.00	44														
	45		Added drill fluid; driller running out of water.												
36.00	46				3-24 3-25	13 16 17	33	67							
	47		Lean CLAY (CL); light brown; moist; medium plasticity, high dry strength, no dilatancy, medium toughness FINES ; very stiff consistency.												
34.00	48		Same.		3-26	5 4 6	10	56							
	49		Elastic SILT (MH).												
32.00	50														
	51		Elastic SILT (MH); light brown; moist; medium plasticity, high dry strength, no dilatancy, low toughness FINES ; stiff consistency.		3-27 3-28	9 13 20	33	100							
30.00	52														
	53		Elastic SILT (MH); light brown; moist; medium plasticity, medium dry strength, no dilatancy, low toughness FINES ; stiff consistency.		3-29	9 9 12	21	89							
28.00	54														
	55														

(continued)

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REPORT TITLE				HOLE ID
BORING RECORD				B-3
DIST. 04	COUNTY SOL	ROUTE	POSTMILE	PROJECT ID 160600
PROJECT OR BRIDGE NAME: Quincy - Stevenson Rd. Bridge				
BRIDGE NUMBER	PREPARED BY K. D.		DATE	SHEET 2 of 5

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55	Elastic SILT (MH) *(continued)*.											
26.00	56												
	57	SILTY SAND (SM); dense; dark yellowish brown; wet; fine, subangular SAND ; 30% SILT.	3-30	26 50		67							
		Well-graded SAND with GRAVEL (SW); very dense; olive gray; wet; fine, rounded to angular GRAVEL ; mostly medium to coarse, subrounded to subangular SAND.	3-31	39 50		80							
24.00	58												
	59	Well-graded GRAVEL with SAND (GW); very dense; olive gray; wet; fine to coarse, subrounded to angular GRAVEL ; medium to coarse SAND.											
22.00	60												
	61	Driller noted hard drilling (chatter).											
20.00	62	GRAVEL.											
	63												
18.00	64												
	65	SILT with SAND (ML); driller noted easier drilling (clay).											
16.00	66												
	67	SILT with SAND (ML); brown; wet; 15% fine SAND ; medium plasticity, low dry strength, slow dilatancy, low toughness FINES ; medium stiff consistency.	3-32	16 21 23	44	89							
14.00	68												
	69												
12.00	70												
	71												
10.00	72												
	73												
8.00	74	Well-graded SAND with GRAVEL (SW); driller noted equipment dropped 2 ft no resistance; possible void at 74 ft.											
	75												
6.00	76												
	77	Well-graded SAND with GRAVEL (SW); very dense; olive gray; wet; mostly coarse, subrounded to subangular SAND ; fine subrounded to subangular SAND.	3-33 3-34	24 50		83							
4.00	78		3-35	10 29 50	79	87							
	79	Well-graded GRAVEL with SAND (GW); very dense; olive gray; wet; some fine to coarse, subrounded to subangular, flat and elongated GRAVEL ; fine to coarse SAND.											
2.00	80												
	81	Switched drill bit to sand / gravel bit 82 ft casing in ground; drilled to 86.5 without casing. CLAY?.											
0.00	82	Lean CLAY (CL); driller indicated hard drilling.											
	83												
-2.00	84												
	85												

(continued)

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REPORT TITLE BORING RECORD				HOLE ID B-3	
DIST. 04	COUNTY SOL	ROUTE	POSTMILE	PROJECT ID 160600	
PROJECT OR BRIDGE NAME Quincy - Stevenson Rd. Bridge					
BRIDGE NUMBER	PREPARED BY K. D.			DATE	SHEET 3 of 5

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		Lean CLAY (CL) *(continued)*.												
-4.00	86														
	87		Lean CLAY (CL); yellowish orange; moist; medium plasticity, medium dry strength, no dilatancy, medium toughness FINES ; hard consistency.		3-36 3-37	5 23 35	58	89							
-6.00	88		Same.		3-38	13 23 27	50	94							
	89														
-8.00	90														
	91														
-10.00	92														
	93														
-12.00	94		Well-graded SAND with GRAVEL (SW); driller noted sand and gravel layer; observe fine angular GRAVEL.												
	95														
-14.00	96														
	97		Well-graded SAND with GRAVEL (SW); very dense; olive gray; 10% fine to coarse, subrounded GRAVEL ; 70% medium, subrounded to subangular SAND ; 20% coarse SAND.		3-39	25 29 30	59	78							
-16.00	98														
	99														
-18.00	100														
	101		Noted larger GRAVEL in cuttings. Fine to coarse GRAVEL, subrounded to angular.												
-20.00	102														
	103														
-22.00	104		SILT with SAND (ML); driller noted out of sand and gravels.												
	105														
-24.00	106														
	107		SILT with SAND (ML); dark yellowish brown; moist; 15% fine SAND ; low plasticity, low dry strength, slow dilatancy, low toughness FINES ; hard consistency.		3-40 3-41	16 32 50	82								
-26.00	108		SILT with SAND (ML); yellowish brown; moist; 20% fine SAND ; low plasticity, low dry strength, rapid dilatancy, low toughness FINES ; hard consistency.		3-42	15 26 27	53								
	109														
-28.00	110														
	111														
-30.00	112														
	113														
-32.00	114		Lean CLAY (CL); driller noted increased drill resistance. Hard / stiff CLAY.												
	115														

(continued)

5 BR - STANDARD 160600-SUBSURFACE DATA CALTRANS TEMPLATE.GPJ CALTRANS LIBRARY (FEB 2013).GLB 10/31/16

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REPORT TITLE				HOLE ID	
BORING RECORD				**B-3**	
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**	
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**					
BRIDGE NUMBER	PREPARED BY **K. D.**			DATE	SHEET **4 of 5**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		Lean CLAY (CL) *(continued)*.												
-34.00	116		Lean CLAY (CL); driller noted transition to SAND.												
	117		Lean CLAY (CL); yellowish brown; moist; medium plasticity, medium dry strength, no dilatancy, medium toughness FINES ; hard consistency.		3-43	24 24 31	55	22							
-36.00	118														
	119														
-38.00	120														
	121														
-40.00	122														
	123														
-42.00	124														
	125														
-44.00	126		SILTY SAND (SM); no liners in sample.												
	127		SILTY SAND (SM); very dense; light brown; moist; fine SAND ; 40% SILT.		3-44	25 36 45	81	78							
-46.00	128		SILT with SAND (ML); light brown; moist; fine SAND ; low plasticity, no dry strength, slow dilatancy, low toughness FINES ; hard consistency.		3-45	18 26 30	56	94							
	129														
-48.00	130														
	131														
-50.00	132														
	133														
-52.00	134														
	135														
-54.00	136														
	137		SILTY SAND (SM); very dense; moist; fine SAND ; 35% SILT.		3-46	41 50									
-56.00	138				3-47	23 30 41	71								
	139		SILT with SAND (ML); light red brown; moist; some fine SAND ; low plasticity, low dry strength, no dilatancy, low toughness FINES ; hard consistency.												
-58.00	140		Bottom of borehole at 139.0 ft bgs												
	141														
-60.00	142		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	143														
-62.00	144														
	145														

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REPORT TITLE **BORING RECORD**				HOLE ID **B-3**
DIST. **04**	COUNTY **SOL**	ROUTE	POSTMILE	PROJECT ID **160600**
PROJECT OR BRIDGE NAME **Quincy - Stevenson Rd. Bridge**				
BRIDGE NUMBER	PREPARED BY **K. D.**		DATE	SHEET **5 of 5**

Appendix E. Kleinfelder (2006) Boring Logs

APPENDIX A
FIELD INVESTIGATION

General

The subsurface conditions at the bridge site were explored on December 27 and 28, 2005 by drilling two borings to a depth of 101½ feet each below existing roadway surface at each end of the bridge. The exploration for the proposed realignment of Stevenson Bridge Road was conducted on March 7, 2006, and consisted of two borings completed to depths of 5 and 11½ feet. All borings were drilled using a Mobile BK-57 truck-mounted drill rig equipped with a seven-inch diameter hollow-stem auger. The locations of borings performed for this investigation are shown on Plate 2 of the report.

Borings were located in the field by visual sighting and/or pacing from existing site features, therefore, the location of borings shown on Plate 2 should be considered approximate and may vary from that indicated on the plate.

Our engineer maintained a log of the borings, visually classified soils encountered according to the Unified Soil Classification System (see Plate A-1) and obtained relatively undisturbed and bulk samples of the subsurface materials. A key to the Logs of Borings is also presented on Plate A-1 of this appendix; Logs of Borings are presented on Plates A-2 through A-5.

Sampling Procedures

Soil samples were obtained from the borings using either a Modified California or Standard Penetration Sampler driven 18 inches (unless otherwise noted) into undisturbed soil using a 30-inch drop of a 140-pound automatic hammer. Blow counts were recorded at six-inch intervals for each sample attempt and are reported on the logs in terms of blows-per-foot for the last foot of penetration. Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance, and returned to our Fairfield laboratory for further testing. After borings were completed, the deeper borings were backfilled with cement grout per the County drill regulations, while the shallower borings were backfilled with the drill cuttings.

LIST OF ATTACHMENTS

The following plates are attached and complete this appendix:

Plate A-1	Unified Soil Classification System/Log Key
Plate A-2	Log of Boring B-1
Plate A-3	Log of Boring B-2
Plate A-4	Log of Boring B-3
Plate A-5	Log of Boring B-4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (Half or more of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (Half or more of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit equal to or greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
VARIABLEY WEATHERED BEDROCK				SANDY SILTSTONE
				SILTSTONE
				SILTSTONE - CLAYSTONE
				CLAYSTONE
				SANDSTONE

LOG KEY SYMBOLS

BULK / BAG SAMPLE

STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)

MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)

SHELBY TUBE (3 inch outside diameter)

CALIFORNIA SAMPLER (3 inch outside diameter)

NO RECOVERY

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKLY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SV	PARTICLE SIZE ANALYSIS
PI	PLASTICITY INDEX	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SS	SOLUBLE SULFATES
P	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
SF	SOIL FERTILITY		

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16" - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2" - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATE A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

KLEINFELDER

Date: 12-1-05 Project Number: 63601
Drawn by: J. Gilbert Filename: USCS/Log Key

UNIFIED SOIL CLASSIFICATION SYSTEM / LOG KEY

STEVENSON BRIDGE
YOLO/SOLANO COUNTIES, CALIFORNIA

PLATE

A-1

Surface Conditions:	Asphalt Road	Date Completed:	12/27/2005
Groundwater:	Groundwater encountered at a depth of approximately 46-1/2 feet below existing site grade during drilling.	Logged By:	P. Sorci
		Total Depth:	Approximately 101-1/2 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 94 feet (msl)

Elevation (ft., msl)	Depth (feet)	FIELD: Sample Type	FIELD: Sample No.	FIELD: Blows/ft	FIELD: Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Liquid Limit	LABORATORY: Plasticity Index	LABORATORY: Passing #4 Sieve (%)	LABORATORY: Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
														Asphalt Concrete: approximately 5"
														SILT (ML): brown, dry, hard, trace of fine sand
			B1-1-B B1-1-A	7								Corrosion: see Appendix		- grades medium stiff
90	5													
			B1-2-B B1-2-A	41	2.75							Corrosion: see Appendix		- grades with trace of Clay, very stiff
85	10													
			B1-3-B B1-3-A	33										- grades with fine Sand, very stiff to hard
80	15													
			B1-4-B B1-4-A	44	2.5	89	13					UC=350 psf @ 2% Strain		- grades with some fine Sand, hard, no cohesion
														Poorly Graded Silty SAND (SP-SM): brown, dry to moist, loose, fine sand, with fines
75	20													
			B1-5-B B1-5-A	9							29			
70	25													
			B1-6-B B1-6-A	54										- grades moist, dense

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/7/06

KLEINFELDER

Drafted By: J. Gilbert
Date: 3/21/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-1

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE

A-2

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
65														
	30		B1-7-B B1-7-A	13								Direct Shear: phi=40°, c=0 psf		- grades with fines, increasing with depth
60	35		B1-8-B B1-8-A	60										- grades with some fine Gravel up to 1/4", subrounded.
55														**Poorly Graded Gravel with trace of Sand (GP):** gray, moist, very dense, fine gravel up to 1/4", subrounded, trace of fine to coarse sand, trace of fines
	40		B1-9-B B1-9-A	83										
50	45		B1-10-B B1-10-A	35										
45														**Lean CLAY (CL):** yellow-brown mottled olive-brown, moist, stiff, some plasticity
	50		B1-11-B B1-11-A	11	1.75	102	25					UC=2936 psf @ 15% Strain		
40	55		B1-12-B B1-12-A	20	2.0			40	21					**Fat CLAY (CH):** yellow-brown mottled olive-brown, moist, very stiff, highly plastic
35	60													

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
			B1-13-B B1-13-A	12	1.0		36							**Lean CLAY (CL):** yellow-brown, moist, medium stiff to stiff, slightly plastic
30														**Poorly Graded SAND with some fines (SP):** brown, moist to wet, medium dense, fine sand, less fines with depth
	65		B1-14-B B1-14-A	23								Direct Shear: phi=33°, c=600 psf		
25														- note sand heaving in auger approximately 2'
	70		B1-15-B B1-15-A	61						100	6			- grades medium Sand with trace of fines
20														**SILT (ML):** yellow-brown, moist, hard
	75		B1-16-B B1-16-A	79										
15														
	80		B1-17-B B1-17-A	20		90	33					UC=1832 psf @ 7% Strain		- grades wet, medium stiff to very stiff
10														
	85		B1-18-B B1-18-A	34										- grades hard
														Silty SAND (SM): brown, wet, hard, fine sand
5														
	90		B1-19-B B1-19-A	34										**Poorly Graded SAND with Gravel (SP):** brown, wet, medium dense to dense, medium sand, fine gravel

KLEINFELDER

LOG OF BORING B-1

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE

A-2

Drafted By: J. Gilbert
Date: 3/21/2006
Project No.: 63601-1
File Number: stevenson bridge

SAC 2004 63601 STEVENSON BRIDGE.GPJ 3/21/06

LOG OF BORING B-1

Elevation (ft., msl)	Depth (feet)	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	DESCRIPTION
0												SILT (ML): brown, moist, hard, moderately cemented
	95	B1-20-B B1-20-A	51									
-5	100	B1-21-B B1-21-A	69									
												Boring completed at a depth of approximately 101-1/2 feet below existing site grade. Upon completion the boring was grouted using neat cement and capped with cold-patch asphalt.
-10	105											
-15	110											
-20	115											
-25	120											
-30												

KLEINFELDER

Drafted By: J. Gilbert
Date: 3/21/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-1

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
4 of 4
A-2

SAC 2004 63601 STEVENSON BRIDGE.GPJ 3/21/06

Surface Conditions:	Asphalt Road	Date Completed:	12/28/2005
Groundwater:	Groundwater encountered at a depth of approximately 50 feet below existing site grade during drilling.	Logged By:	P. Sorci
		Total Depth:	Approximately 101-1/2 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 93 feet (msl)

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No. (FIELD)	Blows/ft (FIELD)	Pocket Penetrometer (tsf) (FIELD)	Dry Density (pcf) (LABORATORY)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
	0													**Asphalt Concrete:** approximately 3"
														SILT with trace of Sand (ML): brown, moist, stiff to very stiff, fine sand
90			B2-1-B B2-1-A	15										
	5													**Sandy SILT (ML):** brown, moist, soft to medium stiff
			B2-2-B B2-2-A	4										
85														**SILT with Sand (ML):** brown, dry to moist, medium stiff
	10		B2-3-B B2-3-A	6										
80														**Fat CLAY (CH):** dark brown, moist, very stiff to hard, high plasticity
	15		B2-4-B B2-4-A	27		114	16					UC=21,953 psf @ 5% Strain		
75														**SILT (ML):** brown, dry to moist, stiff to very stiff
	20		B2-5-B B2-5-A	13										
70														**Lean CLAY (CL):** brown mottled olive-brown, dry to moist, very stiff, some plasticity
	25		B2-6-B B2-6-A	19										
65														

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/27/06

KLEINFELDER

Drafted By: J. Gilbert
Date: 4/27/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-2

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
1 of 4
A-3

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
	30		B2-7-B B2-7-A	33										- grades yellow-brown mottled dark brown, hard
60														
	35		B2-8-B B2-8-A	29										- grades Clay with Gravel, moist, very stiff to hard, fine gravel
55														
	40		B2-9-B B2-9-A	16		114	18					UC=6128 psf @ 5% Strain		- grades Lean Clay, yellow-brown, very stiff, moderate plasticity
50														
	45		B2-10-B B2-10-A	15										- fine Sand interlayer, Clay with moderate plasticity, stiff to very stiff
45														
	50		B2-11-B B2-11-A	15				35	18			∇		- grades wet
40														**Fat CLAY (CH):** brown, moist, very stiff to hard, highly plastic
	55		B2-12-B B2-12-A	32		105	23					UC=6948 psf @ 5% Strain		
35														**Lean CLAY (CL):** yellow-brown, moist, stiff, medium plasticity
	60													

FIELD / LABORATORY

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/27/06

KLEINFELDER

Drafted By: J. Gilbert
Date: 4/27/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-2

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
2 of 4
A-3

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	Description
			B2-13-B B2-13-A	10								Corrosion: see Appendix		
30														
	65		B2-14-B B2-14-A	50								Corrosion: see Appendix		- grades brown to dark brown - grades with trace of fine Gravel
25														**Poorly Graded SAND (SP):** gray to brown, wet, medium sand, note heaving sands
	70		B2-15-B B2-15-A	61								Direct Shear: phi=55°, c=0 psf		
20														
	75		B2-16-B B2-16-A	30						63	3			- grades Gravelly Sand, medium to coarse sand, fine gravel, trace of fines
15														**Lean CLAY (CL):** brown, moist, medium stiff to stiff, low to medium plasticity
	80		B2-17-B B2-17-A	13		93	31					UC=1957 psf @ 6% Strain		
10														**SILT with some Sand (ML):** brown, moist, very stiff, fine sand
	85		B2-18-B B2-18-A	26										
5														**Lean CLAY (CL):** brown, moist, very stiff, moderate plasticity
	90		B2-19-B B2-19-A	19										

KLEINFELDER

Drafted By: J. Gilbert
Date: 4/27/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-2

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
0	95		B2-20-B B2-20-A	47										- grades with trace of fine Gravel up to 1/2"
-5	100		B2-21-B B2-21-A	68										- grades with trace of Silt, low plasticity
														Boring completed at a depth of approximately 101-1/2 feet below existing site grade. Upon completion the boring was grouted using neat cement and capped with cold-patch asphalt.
-10	105													
-15	110													
-20	115													
-25	120													
-30														

KLEINFELDER

LOG OF BORING B-2

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
4 of 4
A-3

Drafted By: J. Gilbert
Date: 4/27/2006
Project No.: 63601-1
File Number: stevenson bridge

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/27/06

Surface Conditions:	Short grass on road shoulder.	Date Completed:	3/7/2006
Groundwater:	No free groundwater encountered.	Logged By:	P. Sorci
		Total Depth:	Approximately 11-1/2 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 93 feet (msl)

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No. (FIELD)	Blows/ft (FIELD)	Pocket Penetrometer (tsf) (FIELD)	Dry Density (pcf) (LABORATORY)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
														Silty with CLAY/Silty CLAY (CL/ML): brown, moist, medium stiff to stiff, very low plasticity
90			3-2-II 3-2-I	5	1.25									
	5		3-5-II 3-5-I	4	0.5									- grades soft
85														
	10		3-10-II 3-10-I	7	1.75									- grades light brown, dry to moist, medium stiff to stiff, non-plastic
														Boring completed at a depth of approximately 11-1/2 feet below existing site grade. Upon completion the boring was backfilled with drill cuttings.
80														
	15													
75														
	20													
70														
	25													
65														

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/27/06

KLEINFELDER

LOG OF BORING B-3

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
1 of 1
A-4

Drafted By: J. Gilbert
Date: 4/27/2006
Project No.: 63601-1
File Number: stevenson bridge

Surface Conditions:	Road shoulder.	Date Completed:	3/7/2006
Groundwater:	No free groundwater encountered.	Logged By:	P. Sorci
		Total Depth:	Approximately 5 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 89 feet (msl)

Elevation (ft., msl)	Depth (feet)	FIELD: Sample Type	FIELD: Sample No.	FIELD: Blows/ft	FIELD: Pocket Penetrometer (tsf)	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Liquid Limit	LABORATORY: Plasticity Index	LABORATORY: Passing #4 Sieve (%)	LABORATORY: Passing #200 Sieve (%)	LABORATORY: Other Tests	Lithography	DESCRIPTION
														Silty CLAY (CL): brown, moist, soft to stiff, low plasticity
			B-4 @ 1-5 4-2-I	4	1.25							R-Value=8		
85	5													Boring completed at a depth of approximately 5 feet below existing site grade. Upon completion the boring was backfilled with drill cuttings.
80	10													
75	15													
70	20													
65	25													

KLEINFELDER

Drafted By: J. Gilbert
Date: 4/27/2006
Project No.: 63601-1
File Number: stevenson bridge

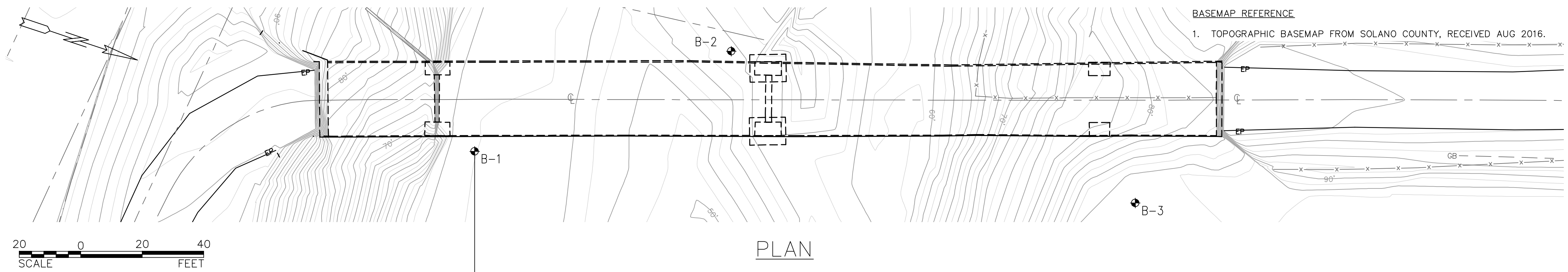
LOG OF BORING B-4

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
1 of 1

A-5

Appendix F. Log of Test Boring Sheets



PLAN

B-1 (3/4)

Elev. 100.0'

SILT (ML); light brown; dry; few fine SAND ; (Alluvium).
Few sub-rounded to sub-angular gravel up to 1 in. in cuttings.
Increased moisture at 4 ft.
7 | 2.5 — SILT (ML); brown variegated with red brown and gray; moist; few fine SAND ; mostly low plasticity, low dry strength, rapid dilatancy fines ; few sand to little in lenses.
8 | 1.4 — SILTY SAND (SM); silty sand lense less than 2 in.
Elastic SILT (MH); elastic silt and lean clay with sand lenses less than 4 in. thick.
13 | 2.5 — SILT with SAND (ML); brown; wet; some fine SAND; mostly low plasticity, rapid dilatancy fines ; gradational contact.
11 | 1.4 — Elastic SILT (MH); brown; moist; few SAND; mostly low to medium plasticity, slow dilatancy fines; lense of light gray, lean clay with sand lens at 10.5 ft.
41 | 2.5 — Lean CLAY (CL); strong brown to brown with variegated light gray; moist; trace SAND; mostly medium plasticity, slow to none dilatancy fines; charcoal at 15 ft., thin lens of elastic silt to elastic silt with sand at 15–15.5 ft.
16 | 1.4
10 | 2.5 — Elastic SILT (MH); brown; moist; few SAND; mostly low to medium plasticity, slow dilatancy fines; gradational contact to ML at 19.75.
9 | 1.4 — SILT (ML); brown; wet; few SAND; mostly low plasticity, low dry strength, rapid dilatancy fines; thin lens of elastic silt between 20.5 to 21.0 ft.
18 | 2.5 — SANDY SILT (ML); brown; wet; some fine SAND; low to none plasticity, rapid dilatancy fines.
15 | 1.4 — Poorly graded SAND with SILT (SP–SM); medium dense; brown gray; wet; fine SAND; few fines.
Elastic SILT (MH); olive brown; iron stains.
Well-graded GRAVEL (GW); fine GRAVEL in the bottom of sample.
Driller indicated fluid loss and coarse sand and sub-angular GRAVEL up to 1/2" in cuttings, thick and drilling fluid.
2.5 — Well-graded GRAVEL with lenses of well graded gravel with sand very dense; dark gray; wet; subangular to rounded up to 2" GRAVEL; little fine to coarse SAND; weak cementation; consisting of chert, greenstone. graywacke.
52 | 1.4 — Well-graded SAND with GRAVEL (SW); very dense; dark brown to gray; wet; few rounded to rounded up to 1/2" GRAVEL; trace fines; weak cementation.
Well-graded GRAVEL (GW); driller indicate large gravels and cobbles with loss of fluid – mixed even thicker fluid. Cobbles at 32.5 ft.
22 | 1.4 — Medium dense; gray; greater than 1.25", rounded GRAVEL; well-graded GRAVEL with cobbles at bottom of sample.
Driller indicated "Out of gravel" change at 36 ft.
Lean CLAY (CL).
Well-graded GRAVEL (GW); thin gravel lens at 37 ft. Less than 6 in. as indicated by sound and cuttings; approx. 8 in. of caving
Well-graded GRAVEL (GW).
Driller indicated sand and gravel (coarse sand and fine gravels in cutting).
Loose as indicated by driller.
GRAVEL softer at 41 ft, more sand, less gravel.
Grades.
SANDY lean CLAY (CL); brown; fine to coarse SAND; in cuttings.
8 ft. of caving at 45 ft, sample interval, continued drilling with sample.
Well-graded SAND (SW); few fine up to 1/4" GRAVEL; medium to coarse SAND; 8 ft of caving at sample interval, large gravel in hole.
Elected to case hole from 0–48.5.
2.5 — Lean CLAY (CL); olive brown; moist; trace fine SAND; medium plasticity fines;
Sharp contact with lens of well-graded SAND with GRAVEL.
46 | 1.4 — Well-graded SAND with GRAVEL (SW); very dense; dark brown gray wet; some fine up to 3/4", rounded GRAVEL; trace medium to very coarse SAND ; trace fines.
Lean CLAY (CL); olive brown grades to light brown; moist; about 1/2", rounded GRAVEL; few very fine SAND; mostly medium plasticity, slow to none dilatancy fines ; isolated rounded GRAVEL at the bottom of sample.
Driller indicated stiff to very stiff CLAY.
30 minutes to drill 51–56.5 ft..
Well-graded SAND (SW); driller indicated SAND at 56.5. Thin lens (1 ft thick).
44 | 2.5 — SANDY SILT (ML); olive brown; moist; some very fine to medium SAND; mostly low plasticity, rapid dilatancy fines; lenses of SILT with SAND.
21 | 1.4

Elevation scale (left): 105, 100, 95, 90, 85, 80, 75, 70, 65, 60, 55, 50, 45, 40

5 0 5 10
SCALE FEET

SILT (ML); olive brown; moist; trace SAND; low plasticity to none, rapid dilatancy fines; grades to SILT.
Well-graded SAND with SILT and GRAVEL (SW–SM); Driller indicated SAND and GRAVEL between 65–68 ft.
1.4 — Well-graded SAND with GRAVEL (SW); very dense; dark brown gray; wet; rounded to subrounded GRAVEL; medium to very coarse SAND; few fines; weak cementation; rounded to subrounded GRAVEL with few angular gravel up to 3/4", consisting of quartz, chert, graywacke.
SILT with SAND (ML); brown; moist; few very fine to fine SAND ; nonplastic to low plasticity, low dry strength, rapid dilatancy fines ; Decrease SAND in cuttings at 77 ft.
59 | 2.5
22 | 1.4 — Elastic SILT (MH); brown; moist; trace SAND; mostly medium plasticity, slow dilatancy fines.
Used 3-7/8" clay bit.
15 minutes to drill 83–88 ft. Driller indicated clay from 81–88 ft.
Assumed contact based on driller indicating slightly sandier and asier drilling conditions.
SILT with SAND (ML); brown; moist; little very fine to fine SAND; low plasticity, rapid dilatancy fines; trace subangular sandstone rock fragments less than 5% in the bottom 6" of sample.
31 | 1.4 — Some SAND.
Thin sand lens less than 6" thick at about 91 ft as incredibly dark.
About 35 minutes to drill from 89.5 to 93 ft. Driller indicated stiff clays.
About 20 minutes to drill from 93–98 ft (~ 4 min/ft). Clayey / silty throughout as indicated by driller.
Sandy silt on the end of drill bit.
71 | 2.5 — SILTY SAND (SM); dense; brown; moist; very fine to fine SAND; little fines; weak cementation; Gradational contact.
24 | 1.4 — Poorly graded SAND (SP); very dense; gray brown; moist; fine to medium SAND; few fines; gradational contact.
Elastic SILT (MH); brown; moist; trace very fine SAND ; low to medium plasticity, rapid to slow dilatancy fines; trace to few very fine sand in lenses less than 1/2 in.
Isolated round sandstone rock fragments less than 1/4 in.
Approx. 1 hour to drill 101 to 109 ft. Clayey throughout with a few thin sandy lenses less than 6 in. as indicated by driller.
Lean CLAY (CL); Assumed contact. Thin lens of lean CLAY in upper 4 in of sample.
39 | 1.4 — Elastic SILT (MH); brown; moist; few very fine SAND ; medium plasticity, slow dilatancy fines.
SILT with SAND (ML); few very fine SAND ; low plasticity, rapid dilatancy fines.
Lean CLAY (CL); Driller indicated stiffer drilling and clayey at about 111.5 ft.
Drilling from 110.5 to 118 ~45 minutes. Clayey throughout.
82 | 2.5 — Lean CLAY (CL); brown; moist; trace very fine SAND ; mostly medium plasticity, none to slow dilatancy fines.
44 | 1.4 — Lean CLAY (CL); brown to olive brown; moist; trace SAND ; medium plasticity fines ; With lens of 5–10% SAND.

Date: 12–14 Sept 2016
Terminated at Elev. –21.0'
Eri = 76.5%

Elevation scale (right): 40, 35, 30, 25, 20, 15, 10, 5, 0, –5, –10, –15, –20, –25

PROFILE

LEGEND OF BORING OPERATIONS

Hole ID; Description of material; Field & Lab Tests; GWS Elev; Date measured; Material change; Estimated material change; Soil/Rock boundary; Location; Top Hole Elev; Casing driven; Size of Sampler (inches); SPT N-Value or CM N-Value (per ASTM 1586-99), P = push sample, or as noted; Boring Date; Terminated at Elev; Hammer Energy Ratio (ER i) = %

ROTARY BORING

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger boring (hollow or solid stem bucket)
Size	R RW RC	Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing
	P	Wire-line rotary percussion boring
Size	R	(Air) Rotary drilled diamond core
Size ●	HD HA	Hand driven (1-inch soil tube) hand auger
	D	Dynamic cone penetration boring
◀	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

Note: Size in inches

LEGEND OF LABORATORY TESTS

- (C) Consolidation (ASTM D 2435)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (M) Moisture Content (ASTM D 2216)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (R) R-Value (CTM 301)
- (UC) Unconfined Compression, Soil (ASTM D 2166) Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)

LEGEND OF EARTH MATERIALS

GRAVEL; SAND; SILT; CLAY; SILTY SAND; CLAYEY SAND; SILTY CLAY; Fat CLAY; Elastic SILT; PEAT and/or ORGANIC MATTER; IGNEOUS ROCK; SEDIMENTARY ROCK; METAMORPHIC ROCK

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CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

SPT N_{60} (Blows/12 in)	Granular	Cohesive
0–4	Very Loose	Very Soft
5–9	Loose	Soft
10–29	Medium Dense	Stiff
30–49	Dense	Very Stiff
≥50	Very Dense	Hard
		Very Hard

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REVISIONS			
NO.	DESCRIPTION	APPROVED BY	DATE

FIELD BOOK NO.

SCALE — HORIZONTAL: AS NOTED — VERTICAL: AS NOTED

DRAWN BY: CHECKED BY:

SUBMITTED R.C.E. No.

SOLANO COUNTY
TRANSPORTATION DEPARTMENT
333 SUNSET AVE. SUITE 230
SUISUN CITY CA 94585
TEL: (707) 421-6069 FAX: (707) 429-2894

APPROVED BY:

DATE:

STEVENSON ROAD BRIDGE
LOG OF TEST BORINGS

B-1

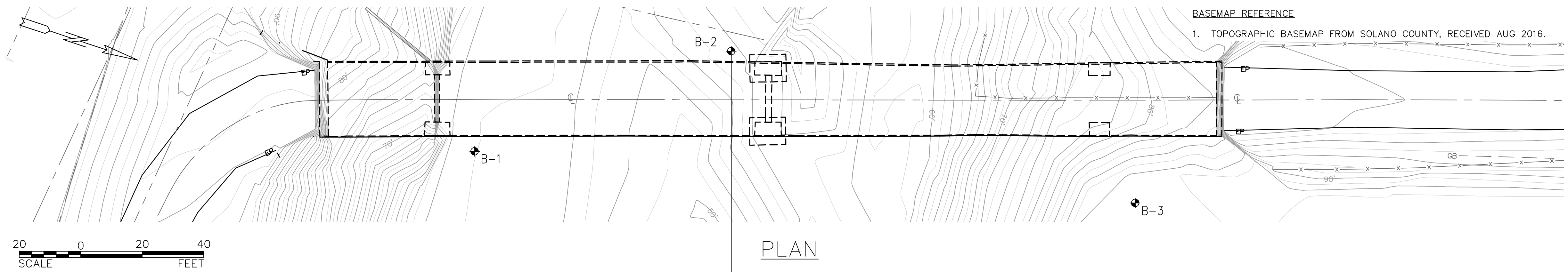
DATE 10/31/16

SHEET 1 OF 3

DWG

BASEMAP REFERENCE

1. TOPOGRAPHIC BASEMAP FROM SOLANO COUNTY, RECEIVED AUG 2016.



PLAN

LEGEND OF BORING OPERATIONS

Top Hole Elev; Location; Hole ID; Description of material; Field & Lab Tests; GWS Elev; Date measured; Material change; Estimated material change; Soil/Rock boundary; Casing driven; Size of Sampler (inches); SPT N-Value or CM N-Value (per ASTM 1586-99), P = push sample, or as noted; Boring Date; Terminated at Elev; Hammer Energy Ratio (ER i) = %

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Size	R	(Air) Rotary drilled diamond core
Size	HD / HA	Hand driven (1-inch soil tube) hand auger
●	D	Dynamic cone penetration boring
◄	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

Note: Size in inches

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- C Consolidation (ASTM D 2435)
- CR Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- CU Consolidated Undrained Triaxial (ASTM D 4767)
- DS Direct Shear (ASTM D 3080)
- M Moisture Content (ASTM D 2216)
- PA Particle Size Analysis (ASTM D 422)
- PI Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- R R-Value (CTM 301)
- UC Unconfined Compression, Soil (ASTM D 2166) Rock (ASTM D 2938)
- UU Unconsolidated Undrained Triaxial (ASTM D 2850)
- UW Unit Weight (ASTM D 4767)

LEGEND OF EARTH MATERIALS

GRAVEL, SAND, SILT, CLAY, SILTY SAND, CLAYEY SAND, SILTY CLAY, Fat CLAY, Elastic SILT, PEAT and/or ORGANIC MATTER, IGNEOUS ROCK, SEDIMENTARY ROCK, METAMORPHIC ROCK

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5–9	Loose	Soft
10–29	Medium Dense	Stiff
30–49	Dense	Very Stiff
≥50	Very Dense	Hard
		Very Hard

PROFILE

B-2 3/4 — Elev. 104.0'

Elev.	N	Size
~98	72	2.0
~97	7	1.4
~93	22	2.0
~92	15	1.4
~90	13	2.0
~88	6	1.4
~85	13	2.0
~84	8	1.4
~80	44	2.5
~79	21	1.4
~75	55	2.5
~74	32	1.4
~70	64	2.5
~69	28	1.4
~65	70	2.5
~64	12	1.4
~60		2.5
~59	54	1.4
~55	72	2.5
~54	35	1.4
~45	81	2.5
~44	24	1.4

SILT (ML); SILT and 2 ft. minus rip-rap. Hollow stem (6") auger used from 0–10 ft. About 1 hour to advance through soil/rip-rap layers to sample depth of 5 ft.
Water at 3 ft at time of drilling.
Driller indicated out of rip-rap zone at 4 ft.
Well-graded SAND with CLAY and GRAVEL (SW-SC); very dense; gray brown; wet; little fine up to 1/2", subrounded GRAVEL; fine to very coarse SAND; little fines.
Well-graded GRAVEL with SILT and SAND (GW-GM); loose; gray brown; wet; fine up to 3/4 in, rounded GRAVEL ; some very fine to very coarse SAND; few fines.
Lean CLAY (CL); brown; moist; trace SAND; medium plasticity, none to slow dilatancy, low toughness fines ; switched to rotary wash at 13 ft using 6" HSA as conductor casing.
Elastic SILT (MH); brown; moist; trace SAND ; medium grades to low plasticity, slow dilatancy fines.
Variegated light blue, gray in brown elastic silt matrix; Thin lean clay lenses less than 2".
SILT with SAND (ML); brown; wet; little very fine SAND ; mostly low plasticity, low dry strength, rapid dilatancy fines ; assumed contact.
SILT with SAND (ML); brown; wet; little very fine to fine SAND ; low plasticity, rapid dilatancy fines.
Well-graded GRAVEL with SAND (GW); dense; brown gray; wet; mostly fine, less than 1/2" GRAVEL ; some fine to very coarse SAND ; trace fines ; Driller indicated gravel at 22 ft.
Rounded to subrounded gravel up to 1.5".
Consisting of quartz and graywacke.
Well-graded SAND with GRAVEL (SW); medium dense; brown gray; wet; mostly fine up to 1/2", rounded GRAVEL ; fine to very coarse SAND ; with isolated trace rounded gravel up to 1".
Loss of circulation at 27 ft, mixed thicker drilling fluid.
Well-graded GRAVEL with SAND (GW/GW); dense; brown gray; wet; coarse up to 2.5 in. GRAVEL ; some medium to very coarse SAND.
Cobbles larger than 2.5 ft at 29 ft.
About 60% well-graded coarse sand at bottom of sample near 31 ft with about 15% fines.
SILT with SAND (ML); brown; wet; 15% very fine to fine SAND ; mostly low plasticity, rapid dilatancy fines ; Driller indicated CLAYEY at 31.5 ft
GRAVELS advance while drilling sample creating disturbed sample with limited recovery betweeen 33–34.5 ft.
Thin lenses less than 4 in. of sandy SILT with about 40% very fine to fine SAND.
Poorly graded SAND (SP); dense; dark brown gray; wet; trace fine up to 1/4 in., isolated rounded GRAVEL ; fine to medium SAND ; trace fines ; approx. contact at 37.5 based on drilling resistance.
Poorly graded SAND (SP); medium dense; dark gray; wet; fine to medium SAND ; trace fines.
Poorly graded SAND with GRAVEL (SP); very dense; dark gray; wet; little fine up to 3/8", subrounded GRAVEL ; medium SAND ; trace fines.
Lean CLAY (CL); Driller indicated stiffer drilling resistance and clayey at 46 ft.
Lean CLAY (CL/CL); olive brown variegated with light blue gray veins; moist; trace very fine SAND ; mostly low plasticity, slow dilatancy fines ; some silt.
Time to drill 51–58 ft is 55 min.
Clayey while drilling, possible thin SAND lenses a few inches thick at various depths based on drilling resistance, easing up briefly.
Driller indicated clay at 58 ft with clay on drill bit.
Poorly graded SAND (SP); dense; gray brown; wet; fine to medium SAND ; trace fines ; isolated rounded to subrounded gravel at top of sample near 58.5 ft.
No recovery of sample.
Assumed SP as above.
Drill time 20 minutes for 61–68 ft.
Driller indicated sandy and easy drilling conditions.

SCALE 5 0 5 10 FEET

Elev.	N	Size
~35	25	1.4
~25	75	2.5
~24	34	1.4
~15	19	1.4
~5	82	2.5
~4	52	1.4
~-5	21	1.4
~-15	88.5	2.5
~-16	48	1.4
~-25	29	1.4

Thin gravel lens (less than 4") at 64 ft.
Thin gravel lens (6") at 66 ft.
Thin gravel lens (3") at 67 ft.
Lean CLAY (CL); Driller noted change and stiffer drilling conditions at 67.5 ft.
Lean CLAY (CL); brown; moist; trace SAND ; medium plasticity, none to slow dilatancy, medium toughness fines.
Drilling time 69.5–78 ft is 20 minutes.
Driller noted varible drilling resistance indicating interbedded clays and sands but primarily clay.
SANDY SILT (ML); Approx. contact based on drilling resistance and increased drilling rate.
SILT grades to SANDY SILT brown; moist; mostly low plasticity, rapid dilatancy fines ; Sand grades from 5–10% to 25–35% very fine with depth in sample.
Poorly graded SAND with SILT (SP-SM); dense; gray brown; wet; very fine to fine SAND ; few fines ; 10% fines grades to trace fines.
SILT (ML); 2 in. silt lens at 80.75 ft, below silt lens medium to coarse sand, gray, trace to none fines.
Lean CLAY (CL); Driller indicated stiff clay at 82 ft. Drilling 81–88 ft in 50 minutes.
Assumed stiff clay based on slow drill rate.
Drilling eased up slightly at 86.5 ft.
Elastic SILT (MH).
Elastic SILT (MH); brown; moist; trace SAND ; mostly medium plasticity, slow dilatancy fines ; Silt lense with low plasticity at bottom 2" of sample.
Drilling 84.5–98 ft in 45 min.
Lean CLAY (CL); Stiff drilling likely clay or elastic silt.
Lean CLAY (CL); Assumed clay based on drilling rate.
SILTY SAND (SM); dense; brown; wet; very fine to fine SAND ; some fines ; Drilling eased up at 97 ft.
Sand grades to SP.
Poorly graded SAND (SP); very dense; dark brown gray; wet; fine to medium SAND ; trace fines.
Poorly graded SAND with thin SANDY SILT lenses dense; dark brown gray with brown lenses; mostly fine to medium SAND ; 10% fines ; Coarse SAND lenses near 101 ft. SILT lenses up to 2 in. thick but most are about 1/2 in. thick.
Drill time for 101–108 ft is 40 minutes.
Drilling resist increased at about 103 ft indicating CLAY.
Elastic SILT (MH).
Elastic SILT (MH); olive brown; wet; trace SAND ; medium plasticity, slow dilatancy fines ; thin CLAY lenses, less than 2 in. thick.
Drill time between 109.5–118 ft is 66 minutes.
Lean CLAY (CL/CL); Very hard drilling resistance between 114–118 ft.
Lean CLAY (CL/CL); olive brown variegated with light gray; moist; trace SAND ; medium plasticity, slow to none dilatancy fines ; discontinous lenses.
Lean CLAY (CL/CL); drilling time 121–128 ft is 55 min.
Hard drilling, resist throughout run with possible very thin sandy beds.
Lean CLAY (CL/CL); brown gray; wet; trace SAND ; medium plasticity, slow to none dilatancy fines.

Date: 21–22 Sept 2016
Terminated at Elev. -25.5'
Eri = 76.5%

PROFILE

REVISIONS	NO.	DESCRIPTION	APPROVED BY	DATE

FIELD BOOK NO. | SCALE HORIZONTAL: AS NOTED VERTICAL: AS NOTED | DRAWN BY: CHECKED BY: SUBMITTED R.C.E. No.

SOLANO COUNTY
TRANSPORTATION DEPARTMENT
333 SUNSET AVE. SUITE 230
SUISUN CITY CA 94585
TEL: (707) 421-6069 FAX: (707) 429-2894

APPROVED BY: DATE:

STEVENSON ROAD BRIDGE
LOG OF TEST BORINGS
B-2

DATE 10/31/16
SHEET 2 OF 3
DWG

LEGEND OF BORING OPERATIONS

- Hole ID
- Description of material
- Field & Lab Tests
- GWS Elev
- Date measured
- Material change
- Estimated material change
- Soil/Rock boundary
- Location
- Top Hole Elev
- Casing driven
- Size of Sampler (inches)
- SPT N-Value or CM N-Value (per ASTM 1586-99), P = push sample, or as noted
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BASEMAP REFERENCE

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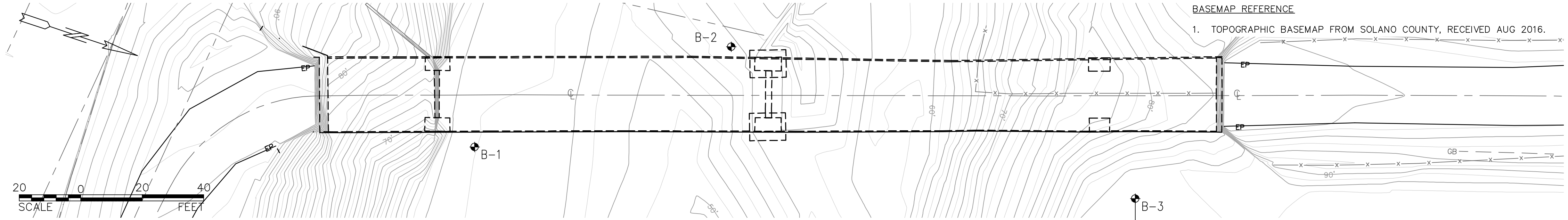

PLAN

BORING LOG B–3
WILL BE COMPLETED FOR FINAL REPORT.

PROFILE

REVISIONS				FIELD BOOK NO.	SCALE			SOLANO COUNTY	APPROVED BY:		
NO.	DESCRIPTION	APPROVED BY	DATE		HORIZONTAL: AS NOTED	DRAWN BY:	CHECKED BY:	TRANSPORTATION DEPARTMENT		STEVENSON ROAD BRIDGE LOG OF TEST BORINGS	DATE 10/31/16
					VERTICAL: AS NOTED	SUBMITTED	R.C.E. No.	333 SUNSET AVE. SUITE 230 SUISUN CITY CA 94585 TEL: (707) 421-6069 FAX: (707) 429-2894	DATE:	B–3	SHEET 3 OF 3
											DWG

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Appendix G. Laboratory Test Results

Appendix H. Analyses and Calculations

CE&G
CAL ENGINEERING & GEOLOGY

DOCUMENT REVIEW COVER SHEET

1. PROJECT NAME Stevenson Road Bridge Design	2. PROJECT NUMBER 160600
3. DOCUMENT TITLE ARS Curve Calculation Package	
4. DOCUMENT STATUS DESIGNATION ☐ Preliminary ☒ Final ☐ Cancelled	
5. NOTES/COMMENTS The purpose of this calculation package is to provide the methods for determining the design ARS curve. The method includes calculating correlated shear wave velocities from SPT values from borings completed by CEG and existing boring information from Kleinfelder's investigation in 2006. These shear wave velocities are then averaged to run Caltrans ARS tool to determine the design probabilistic/deterministic curve .	

ATTACHMENTS	TOTAL NO. OF PAGES
Caltrans ARS tool Output(Spectrum and tabular data)	8
Shear Wave Velocity Calculations Based On SPT Correlations	6
USGS Deaggregation	1
Caltrans Design Spectrum Methodology Article Outlining Correlations (Partial Document)	6

RECORD OF REVISIONS

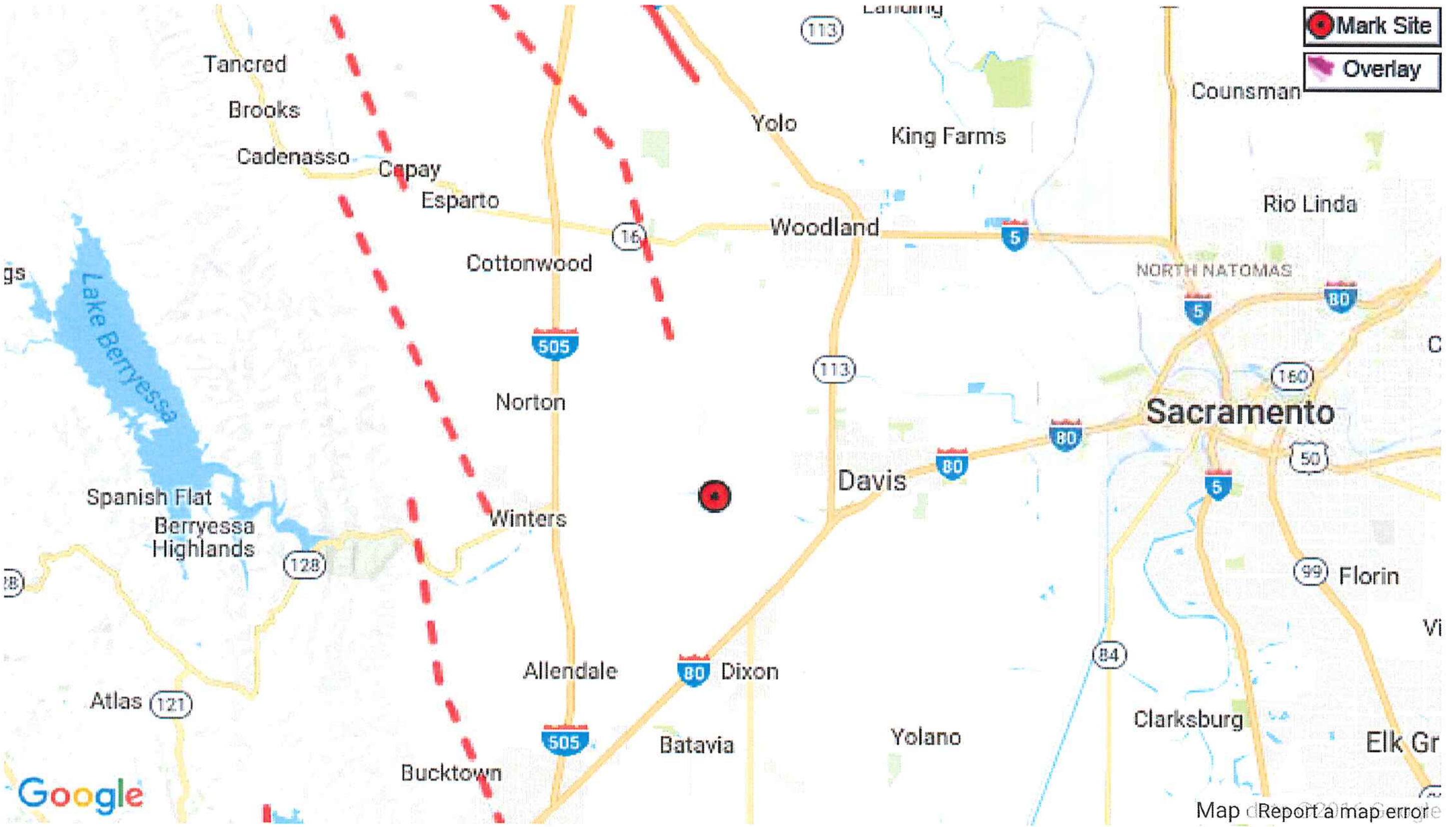
6. No.	7. Reason For Revision	8. Tot. Pgs	10. Originator (Print/Sign/Date)	11. Checker (Print/Sign/Date)	12. QA/QC (Print/Sign/Date)	13. Apprvd./Accptd (Print/Sign)	14. Date (M/D/YY)
1	Initial Issue	21	Mehal Vitthal 10/26/16	Chris Hockett 10/26/16	Mark Myers 10/27/16	Phil Gregory 10-28-16	10/26/16

CALIFORNIA DEPARTMENT OF
TRANSPORTATION

Caltrans ARS Online (v2.3.07)

This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in *Appendix B of Caltrans Seismic Design Criteria*. More...

SELECT SITE LOCATION

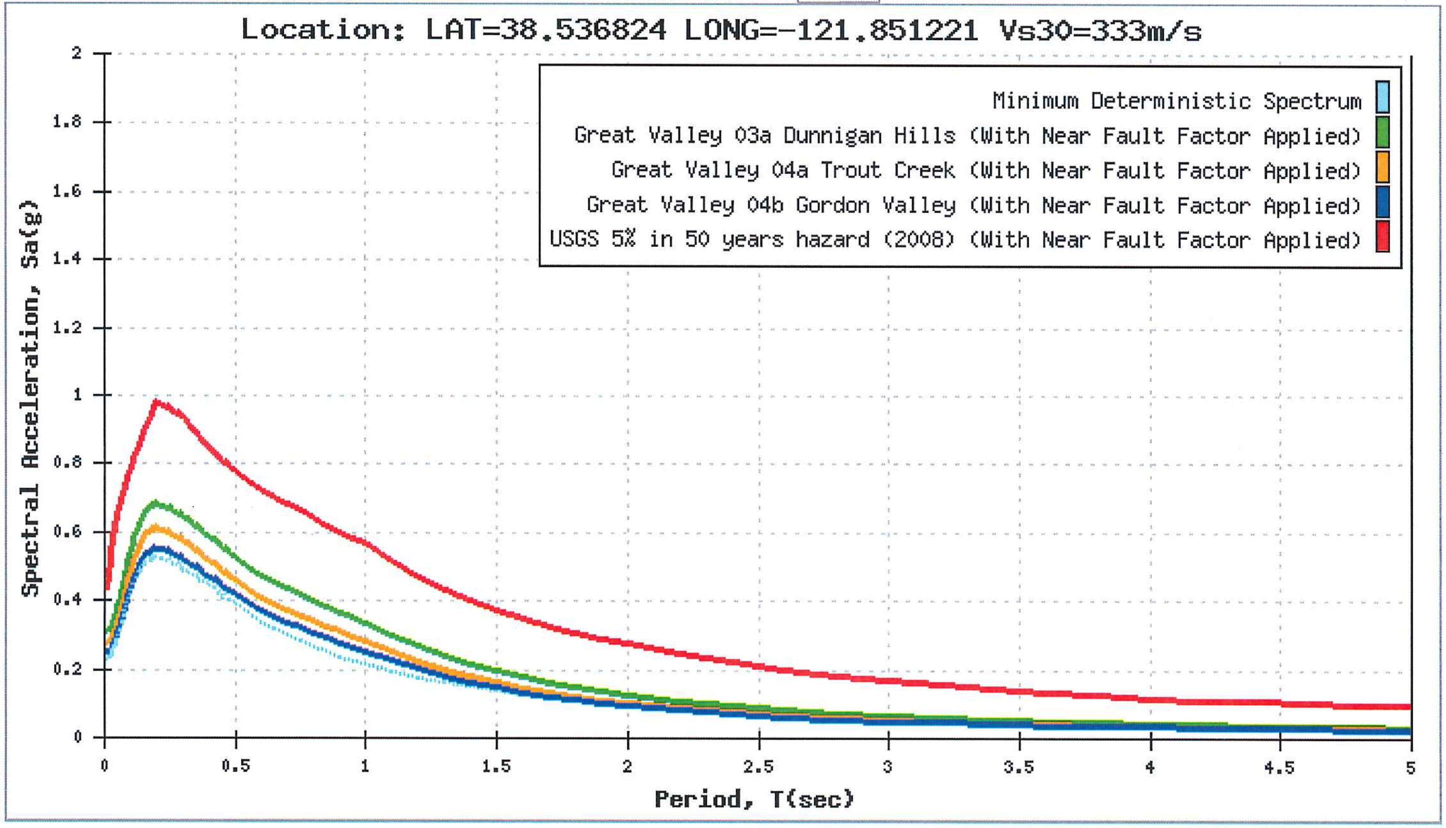


Latitude: 38.536824 **Longitude:** -121.851221 ***Vs*30:** 333 m/s Calculate

CF MV

CALCULATED SPECTRA

Display Curves: 3



Tabular Data | Envelope Only | Hide Near Fault | Axis Scale | Show Basin

Apply Near Fault Adjustment To:

NOTE: Caltrans SDC requires application of a Near Fault Adjustment factor for sites less than 25 km (Rrup) from the causative fault.

☑ Deterministic Spectrum Using

10.39	Km Great Valley 03a Dunnigan Hills
15.79	Km Great Valley 04a Trout Creek
19.32	Km Great Valley 04b Gordon Valley

☑ Probabilistic Spectrum Using

10.39	Km (Recommend Performing Deaggregation To Verify)

RTM

SITE DATA (ARS Online Version 2.3.07)

Shear Wave Velocity, *Vs30*:	333 m/s
Latitude:	38.536824
Longitude:	-121.851221
Depth to Vs = 1.0 km/s:	N/A
Depth to Vs = 2.5 km/s:	3.25 km

DETERMINISTIC

Great Valley 03a Dunnigan Hills	
Fault ID:	95
Maximum Magnitude (MMax):	6.4
Fault Type:	Rev
Fault Dip:	20 Deg
Dip Direction:	E
Bottom of Rupture Plane:	6.00 km
Top of Rupture Plane(Ztor):	3.00 km
Rrup	10.39 km
Rjb:	9.95 km
Rx:	0.21 km
Fnorm:	0
Frev:	1

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.304	1.025	1.000	0.312
0.05	0.372	1.022	1.000	0.381
0.1	0.536	1.020	1.000	0.547
0.15	0.639	1.021	1.000	0.652
0.2	0.668	1.022	1.000	0.683
0.25	0.653	1.024	1.000	0.669
0.3	0.630	1.025	1.000	0.646
0.4	0.572	1.027	1.000	0.587

0.5	0.505	1.044	1.000	0.528
0.6	0.436	1.048	1.040	0.475
0.7	0.382	1.052	1.080	0.434
0.85	0.317	1.054	1.140	0.381
1	0.267	1.055	1.200	0.338
1.2	0.212	1.056	1.200	0.268
1.5	0.155	1.057	1.200	0.196
2	0.098	1.059	1.200	0.125
3	0.052	1.062	1.200	0.066
4	0.034	1.064	1.200	0.043
5	0.025	1.067	1.200	0.032

Great Valley 04a Trout Creek	
Fault ID:	101
Maximum Magnitude (MMax):	6.5
Fault Type:	Rev
Fault Dip:	20 Deg
Dip Direction:	W
Bottom of Rupture Plane:	14.10 km
Top of Rupture Plane(Ztor):	9.00 km
Rrup	15.79 km
Rjb:	12.97 km
Rx:	12.00 km
Fnorm:	0
Frev:	1

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.270	1.026	1.000	0.277
0.05	0.329	1.024	1.000	0.337
0.1	0.478	1.022	1.000	0.488
0.15	0.573	1.023	1.000	0.586
0.2	0.596	1.024	1.000	0.611
0.25	0.581	1.025	1.000	0.596

0.3	0.559	1.026	1.000	0.574
0.4	0.502	1.028	1.000	0.516
0.5	0.439	1.045	1.000	0.459
0.6	0.377	1.049	1.037	0.410
0.7	0.329	1.052	1.074	0.372
0.85	0.272	1.054	1.129	0.323
1	0.227	1.055	1.184	0.284
1.2	0.179	1.056	1.184	0.224
1.5	0.130	1.057	1.184	0.162
2	0.081	1.059	1.184	0.102
3	0.042	1.062	1.184	0.053
4	0.027	1.064	1.184	0.035
5	0.020	1.067	1.184	0.026

Great Valley 04b Gordon Valley	
Fault ID:	104
Maximum Magnitude (MMax):	6.7
Fault Type:	Rev
Fault Dip:	20 Deg
Dip Direction:	W
Bottom of Rupture Plane:	14.10 km
Top of Rupture Plane(Ztor):	9.00 km
Rrup	19.32 km
Rjb:	17.10 km
Rx:	17.10 km
Fnorm:	0
Frev:	1

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.244	1.027	1.000	0.251
0.05	0.297	1.024	1.000	0.304
0.1	0.430	1.023	1.000	0.440
0.15	0.518	1.024	1.000	0.530

0.2	0.540	1.025	1.000	0.553
0.25	0.527	1.026	1.000	0.541
0.3	0.509	1.027	1.000	0.522
0.4	0.456	1.029	1.000	0.469
0.5	0.402	1.045	1.000	0.420
0.6	0.348	1.049	1.023	0.373
0.7	0.306	1.052	1.045	0.337
0.85	0.255	1.054	1.080	0.291
1	0.216	1.055	1.114	0.253
1.2	0.172	1.056	1.114	0.202
1.5	0.126	1.057	1.114	0.148
2	0.080	1.059	1.114	0.095
3	0.042	1.062	1.114	0.050
4	0.028	1.064	1.114	0.033
5	0.021	1.067	1.114	0.025

PROBABILISTIC

Probabilistic Model USGS Seismic Hazard Map(2008) 975 Year Return Period				
Period	**SA(Base Spectrum)**	**Basin Factor**	**Near Fault Factor(Applied)**	**SA(Final Spectrum)**
0.01	0.419	1.027	1.000	0.431
0.05	0.637	1.026	1.000	0.654
0.1	0.763	1.025	1.000	0.782
0.15	0.870	1.025	1.000	0.892
0.2	0.956	1.025	1.000	0.979
0.25	0.933	1.026	1.000	0.957
0.3	0.915	1.027	1.000	0.940
0.4	0.815	1.036	1.000	0.844
0.5	0.745	1.043	1.000	0.777
0.6	0.663	1.047	1.040	0.722
0.7	0.601	1.051	1.080	0.682

0.85	0.516	1.053	1.140	0.620
1	0.449	1.054	1.200	0.568
1.2	0.371	1.055	1.200	0.469
1.5	0.293	1.057	1.200	0.372
2	0.217	1.058	1.200	0.275
3	0.130	1.062	1.200	0.166
4	0.091	1.064	1.200	0.116
5	0.073	1.067	1.200	0.094

MINIMUM DETERMINISTIC SPECTRUM

Period	SA
0.01	0.229
0.05	0.285
0.1	0.422
0.15	0.506
0.2	0.527
0.25	0.512
0.3	0.492
0.4	0.443
0.5	0.392
0.6	0.338
0.7	0.297
0.85	0.249
1	0.213
1.2	0.175
1.5	0.136
2	0.094
3	0.055
4	0.037
5	0.028

Envelope Data

Period	SA
0.01	0.431
0.05	0.654
0.1	0.782
0.15	0.892
0.2	0.979
0.25	0.957
0.3	0.940
0.4	0.844
0.5	0.777
0.6	0.722
0.7	0.682
0.85	0.620
1	0.568
1.2	0.469
1.5	0.372
2	0.275
3	0.166
4	0.116
5	0.094

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Project: STEVENSON BRIDGE Project No.: 160600 Sheet: 1 of 1
Item: AVERAGE SHEAR WAVE VELOCITY, $\overline{V}_{S30}$ Designer: HOCKETT Date: 10/26/16
Checker: Date:

MV
CJt

	323 m/s
	330 m/s
	343 m/s
	365 m/s
+	307 m/s
	1668 m/s

$$\overline{V}_{S30} = \frac{1,668 \text{ m/s}}{5} = 333 \text{ m/s} \rightarrow \text{TO ARS TOOL}$$

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Cal Engineering & Geology

Stevenson Road Bridge
Yolo/Solano County, CA
Approx. Shear Wave Velocities, V30
CEG (2016) Borings

20 October 2016
By Mehal Vitthal

Point ID	Depth (ft)	Field SPT Blows (N)	Depth (m)	E_m	C_B	C_S	C_R	N_{60}	Approx. Unit Weight** (pcf)	Effective Overburder Stress (psf)	$N_{1,60}$	Soil Type	Sands* (SN) (m/s)	Silts* (SL) (m/s)	Cohesive* (C) (m/s)	Layer Thickness (m)	$V_{s\,layer}$ (m/s)	$V_{s\,30}$ (m/s)
B-1	4.5	4	1.37	0.77	1.00	1.00	0.75	4	110	495	8	SL	-		-	4.0	289	**323**
B-1	6.0	8	1.83	0.77	1.00	1.00	0.75	8	113	571	14	SL	-	274	-			
B-1	8.0	8	2.44	0.77	1.00	1.00	0.75	8	112	670	14	SL	-	285	-			
B-1	9.5	11	2.90	0.77	1.00	1.00	0.75	11	115	749	17	SL	-	308	-			
B-1	13.0	26	3.96	0.77	1.00	1.00	0.85	28	125	968	40	C	-	-	310	1.5	310	
B-1	14.5	16	4.42	0.77	1.00	1.00	0.85	17	125	1062	24	C	-	-	310			
B-1	18.0	6	5.49	0.77	1.00	1.00	0.85	7	110	1229	9	SL	-	320	-	2.0	348	
B-1	19.5	9	5.94	0.77	1.00	1.00	0.85	10	112	1303	12	SL	-	346	-			
B-1	23.0	11	7.01	0.77	1.00	1.00	0.95	14	115	1487	16	SL	-	379	-			
B-1	24.5	15	7.47	0.77	1.00	1.00	0.95	18	125	1581	20	SN	380	-	-	7.3	380	
B-1	28.0	32	8.53	0.77	1.00	1.00	0.95	38	132	1825	40	SN	380	-	-			
B-1	29.0	52	8.84	0.77	1.00	1.00	0.95	63	140	1902	65	SN	380	-	-			
B-1	33.0	22	10.06	0.77	1.00	1.00	1.00	28	130	2173	27	SN	380	-	-			
B-1	48.5	32	14.78	0.77	1.00	1.00	1.00	40	130	3220	32	C	-	-	310	2.9	310	
B-1	49.5	46	15.09	0.77	1.00	1.00	1.00	59	133	3291	46	C	-	-	310			
B-1	58.0	28	17.68	0.77	1.00	1.00	1.00	35	125	3823	26	SL	-	380	-	12.2	380	
B-1	59.5	21	18.14	0.77	1.00	1.00	1.00	27	115	3902	19	SL	-	380	-			
B-1	68.0	54	20.73	0.77	1.00	1.00	1.00	69	130	4477	46	SL	-	380	-			
B-1	78.0	37	23.77	0.77	1.00	1.00	1.00	47	130	5153	30	SL	-	380	-			
B-1	79.5	22	24.23	0.77	1.00	1.00	1.00	28	115	5232	17	SL	-	380	-			
B-1	88.0	31	26.82	0.77	1.00	1.00	1.00	40	123	5747	23	SL	-	380	-			
B-1	98.0	45	29.87	0.77	1.00	1.00	1.00	57	130	6423	32	SN	380	-	-	0.5	380	
B-1	99.5	24	30.33	0.77	1.00	1.00	1.00	31	115	6502	17	SL	-	380	-	2.9	380	
B-1	109.0	39	33.22	0.77	1.00	1.00	1.00	50	130	7144	26	C	-	-	310			
B-1	118.0	52	35.97	0.77	1.00	1.00	1.00	66	130	7752	33	C	-	-	310			
B-1	119.5	44	36.42	0.77	1.00	1.00	1.00	56	130	7854	28	C	-	-	310			

* Shear wave velocity correlations from Appendix in Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations" November 2012. Based on 2010 UCLA study by S. Brandenberg.

** Approximate unit weights determined by SPT correlations in Caltrans Geotechnical Manual March 2014

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Stevenson Road Bridge
Yolo/Solano County, CA
Approx. Shear Wave Velocities, V30

CEG (2016) Borings

20 October 2016
By Mehal Vitthal

Point ID	Depth (ft)	Field SPT Blows (N)	Depth (m)	E_m	C_B	C_S	C_R	N_{60}	Approx. Unit Weight** (pcf)	Effective Overburder Stress (psf)	$N_{1,60}$	Soil Type	Sands* (SN) (m/s)	Silts* (SL) (m/s)	Cohesive* (C) (m/s)	Layer Thickness (m)	$V_{s\ layer}$ (m/s)	$V_{s\ 30}$ (m/s)
B-2	5.0	45	1.52	0.77	1.00	1.00	0.75	43	125	313	110	SN	318	-	-	3.0	299	330
B-2	6.5	7	1.98	0.77	1.00	1.00	0.75	7	113	389	15	SN	280	-	-			
B-2	10.0	14	3.05	0.77	1.00	1.00	0.75	13	125	608	24	C	-	-	205	0.9	208	
B-2	11.5	15	3.51	0.77	1.00	1.00	0.75	14	125	702	24	C	-	-	211			
B-2	13.0	8	3.96	0.77	1.00	1.00	0.85	9	112	776	14	SL	-	302	-	3.0	308	
B-2	14.5	6	4.42	0.77	1.00	1.00	0.85	7	95	825	10	SL	-	289	-			
B-2	18.0	8	5.49	0.77	1.00	1.00	0.85	9	110	992	13	SL	-	319	-			
B-2	19.5	8	5.94	0.77	1.00	1.00	0.85	9	110	1063	12	SL	-	323	-			
B-2	23.0	28	7.01	0.77	1.00	1.00	0.95	34	130	1300	42	SN	380	-	-	2.0	380	
B-2	24.5	21	7.47	0.77	1.00	1.00	0.95	25	127	1397	30	SN	380	-	-			
B-2	28.0	35	8.53	0.77	1.00	1.00	0.95	42	138	1661	46	SN	380	-	-			
B-2	29.5	32	8.99	0.77	1.00	1.00	0.95	39	125	1755	41	SL	-	380	-	2.6	380	
B-2	33.0	40	10.06	0.77	1.00	1.00	1.00	51	128	1985	52	SL	-	380	-			
B-2	34.5	28	10.52	0.77	1.00	1.00	1.00	36	128	2083	35	SL	-	380	-			
B-2	38.0	44	11.58	0.77	1.00	1.00	1.00	56	138	2348	52	SN	380	-	-	3.0	380	
B-2	39.5	12	12.04	0.77	1.00	1.00	1.00	15	125	2442	14	SN	380	-	-			
B-2	43.0	32	13.11	0.77	1.00	1.00	1.00	41	130	2678	35	SN	380	-	-			
B-2	44.0	54	13.41	0.77	1.00	1.00	1.00	69	140	2756	59	SN	380	-	-			
B-2	48.0	45	14.63	0.77	1.00	1.00	1.00	58	133	3038	47	C	-	-	310	3.0	309	
B-2	49.5	35	15.09	0.77	1.00	1.00	1.00	45	131	3141	36	C	-	-	308			
B-2	58.0	51	17.68	0.77	1.00	1.00	1.00	65	138	3784	47	SN	380	-	-	3.0	380	
B-2	59.5	24	18.14	0.77	1.00	1.00	1.00	31	125	3878	22	SN	380	-	-			
B-2	68.0	25	20.73	0.77	1.00	1.00	1.00	32	130	4452	21	C	-	-	275	3.0	275	
B-2	78.0	47	23.77	0.77	1.00	1.00	1.00	60	123	5058	38	SL	-	380	-	0.5	380	
B-2	79.5	34	24.23	0.77	1.00	1.00	1.00	43	130	5160	27	C	-	-	305	5.6	278	
B-2	88.0	19	26.82	0.77	1.00	1.00	1.00	24	128	5717	14	C	-	-	251			
B-2	98.0	52	29.87	0.77	1.00	1.00	1.00	66	124	6333	37	SN	380	-	-	3.0	380	
B-2	99.5	52	30.33	0.77	1.00	1.00	1.00	66	124	6426	37	SN	380	-	-			
B-2	108.0	21	32.92	0.77	1.00	1.00	1.00	27	115	6873	14	SL	-	380	-			
B-2	118.0	56	35.97	0.77	1.00	1.00	1.00	71	130	7549	37	C	-	-	310			
B-2	119.5	48	36.42	0.77	1.00	1.00	1.00	61	130	7650	31	C	-	-	310			
B-2	128.0	29	39.01	0.77	1.00	1.00	1.00	37	130	8225	18	C	-	-	289			

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Stevenson Road Bridge
Yolo/Solano County, CA
Approx. Shear Wave Velocities, V30
CEG (2016) Borings

Point ID	Depth (ft)	Field SPT Blows (N)	Depth (m)	E_m	C_B	C_S	C_R	N_{60}	Approx. Unit Weight** (pcf)	Effective Overburder Stress (psf)	$N_{1,60}$	Soil Type	Sands* (SN) (m/s)	Silts* (SL) (m/s)	Cohesive* (C) (m/s)	Layer Thickness (m)	$V_{s\ layer}$ (m/s)	$V_{s\ 30}$ (m/s)
B-3	5	8	1.52	0.77	1.00	1.00	0.75	8	112	560	15	SL	-	274	-	3.2	295	343
B-3	6.5	13	1.98	0.77	1.00	1.00	0.75	13	116	734	21	SL	-	316	-			
B-3	10.5	8	3.20	0.77	1.00	1.00	0.75	8	120	1214	10	SN	372	-	-	0.5	372	
B-3	12	11	3.66	0.77	1.00	1.00	0.75	11	112	1382	13	SL	-	355	-	1.4	355	
B-3	16.5	12	5.03	0.77	1.00	1.00	0.85	13	122	1650	14	SN	380	-	-	4.1	380	
B-3	18	13	5.49	0.77	1.00	1.00	0.85	14	124	1743	15	SN	380	-	-			
B-3	21.5	15	6.55	0.77	1.00	1.00	0.95	18	125	1962	19	SN	380	-	-			
B-3	23	21	7.01	0.77	1.00	1.00	0.95	26	128	2060	25	SN	380	-	-			
B-3	26.5	20	8.08	0.77	1.00	1.00	0.95	24	126	2283	22	SN	380	-	-			
B-3	28	32	8.53	0.77	1.00	1.00	0.95	39	132	2387	36	SN	380	-	-			
B-3	30	28	9.14	0.77	1.00	1.00	1.00	36	130	2522	32	C	-	-	285	1.5	297	
B-3	31.5	35	9.60	0.77	1.00	1.00	1.00	45	130	2624	39	C	-	-	309			
B-3	35	32	10.67	0.77	1.00	1.00	1.00	40	132	2867	34	SN	380	-	-	2.0	380	
B-3	36.5	28	11.13	0.77	1.00	1.00	1.00	36	130	2969	29	SN	380	-	-			
B-3	41.5	25	12.65	0.77	1.00	1.00	1.00	32	130	3307	25	C	-	-	274	3.0	260	
B-3	43	33	13.11	0.77	1.00	1.00	1.00	42	130	3408	32	C	-	-	303			
B-3	46.5	21	14.17	0.77	1.00	1.00	1.00	27	130	3645	20	C	-	-	259			
B-3	48	10	14.63	0.77	1.00	1.00	1.00	13	120	3731	9	C	-	-	203			
B-3	51.5	21	15.70	0.77	1.00	1.00	1.00	27	115	3915	19	SL	-	380	-	1.5	380	
B-3	53	21	16.15	0.77	1.00	1.00	1.00	27	115	3994	19	SL	-	380	-			
B-3	56.5	47	17.22	0.77	1.00	1.00	1.00	61	134	4245	42	SN	380	-	-	3.0	380	
B-3	57.5	75	17.53	0.77	1.00	1.00	1.00	96	140	4322	65	SN	380	-	-			
B-3	66.5	44	20.27	0.77	1.00	1.00	1.00	56	125	4886	36	SL	-	380	-	3.0	380	
B-3	76.5	47	23.32	0.77	1.00	1.00	1.00	60	140	5662	36	SN	380	-	-	3.0	380	
B-3	77.5	79	23.62	0.77	1.00	1.00	1.00	101	140	5739	60	SN	380	-	-			
B-3	86.5	37	26.37	0.77	1.00	1.00	1.00	47	130	6348	26	C	-	-	310	3.0	310	
B-3	88	50	26.82	0.77	1.00	1.00	1.00	64	130	6449	36	C	-	-	310			
B-3	96.5	59	29.41	0.77	1.00	1.00	1.00	76	135	7066	40	SN	380	-	-	3.0	380	
B-3	106.5	52	32.46	0.77	1.00	1.00	1.00	66	125	7692	34	SL	-	380	-			
B-3	108	53	32.92	0.77	1.00	1.00	1.00	68	125	7786	34	SL	-	380	-			
B-3	116.5	55	35.51	0.77	1.00	1.00	1.00	71	130	8361	35	C	-	-	310			
B-3	126.5	51	38.56	0.77	1.00	1.00	1.00	65	132	9057	31	SN	380	-	-			
B-3	128	56	39.01	0.77	1.00	1.00	1.00	72	122	9146	34	SL	-	380	-			
B-3	136.5	57	41.61	0.77	1.00	1.00	1.00	74	132	9738	33	SN	380	-	-			
B-3	137.5	71	41.91	0.77	1.00	1.00	1.00	91	125	9800	41	SL	-	380	-			

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Stevenson Road Bridge
Yolo/Solano County, CA
Approx. Shear Wave Velocities, V30
Kleinfelder (2006) Borings

14 October 2016
By Mehal Vitthal

Point ID	Depth (ft)	Field SPT Blows (N)	Depth (m)	E_m	C_B	C_S	C_R	N_{60}	Approx. Unit Weight** (pcf)	Effective Overburder Stress (psf)	$N_{1,60}$	Soil Type	Sands* (SN) (m/s)	Silts* (SL) (m/s)	Cohesive* (C) (m/s)	Layer Thickness (m)	$V_{s\ layer}$ (m/s)	$V_{s\ 30}$ (m/s)
K_B-1	1.0	4	0.30	0.73	1.00	1.00	0.75	4	113	113	16	SL	-		-			
K_B-1	5.0	25	1.52	0.73	1.00	1.00	0.75	22	123	605	41	SL	-	336	-	6.1	364	
K_B-1	10.0	20	3.05	0.73	1.00	1.00	0.75	18	117	1190	23	SL	-	378	-			
K_B-1	15.0	26	4.57	0.73	1.00	1.00	0.85	27	**101**	1695	30	SL	-	380	-			
K_B-1	20.0	5	6.10	0.73	1.00	1.00	0.95	6	115	2270	6	SN	380	-	-			
K_B-1	25.0	32	7.62	0.73	1.00	1.00	0.95	37	130	2920	31	SN	380	-	-			
K_B-1	30.0	8	9.14	0.73	1.00	1.00	1.00	9	110	3470	7	SN	380	-	-			
K_B-1	35.0	36	10.67	0.73	1.00	1.00	1.00	44	130	4120	31	SN	380	-	-	9.1	380	
K_B-1	40.0	50	12.19	0.73	1.00	1.00	1.00	61	135	4483	40	SN	380	-	-			
K_B-1	45.0	21	13.72	0.73	1.00	1.00	1.00	26	125	4796	16	SN	380	-	-			
K_B-1	50.0	7	15.24	0.73	1.00	1.00	1.00	8	**127**	5119	5	C	-	-	310			**365**
K_B-1	55.0	12	16.76	0.73	1.00	1.00	1.00	15	123	5422	9	C	-	-	310	4.6	310	
K_B-1	60.0	7	18.29	0.73	1.00	1.00	1.00	9	118	5700	5	C	-	-	310			
K_B-1	65.0	14	19.81	0.73	1.00	1.00	1.00	17	115	5963	10	SN	380	-	-	3.0	380	
K_B-1	70.0	37	21.34	0.73	1.00	1.00	1.00	45	122	6261	25	SN	380	-	-			
K_B-1	75.0	47	22.86	0.73	1.00	1.00	1.00	58	121	6554	32	SL	-	380	-			
K_B-1	80.0	12	24.38	0.73	1.00	1.00	1.00	15	**120**	6842	8	SL	-	380	-	4.6	380	
K_B-1	85.0	20	25.91	0.73	1.00	1.00	1.00	25	115	7105	13	SL	-	380	-			
K_B-1	90.0	20	27.43	0.73	1.00	1.00	1.00	25	116	7373	13	SN	380	-	-	1.5	380	
K_B-1	95.0	31	28.96	0.73	1.00	1.00	1.00	37	115	7636	19	SL	-	380	-	1.5	380	
K_B-1	100.0	41	30.48	0.73	1.00	1.00	1.00	50	120	7924	25	SL	-	380	-			

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** Approximate unit weights determined by SPT correlations in Caltrans Geotechnical Manual March 2014. Values in **BOLD** are measured densities

Stevenson Road Bridge
Yolo/Solano County, CA
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Kleinfelder (2006) Borings

CE&G Cal Engineering & Geology

14 October 2016
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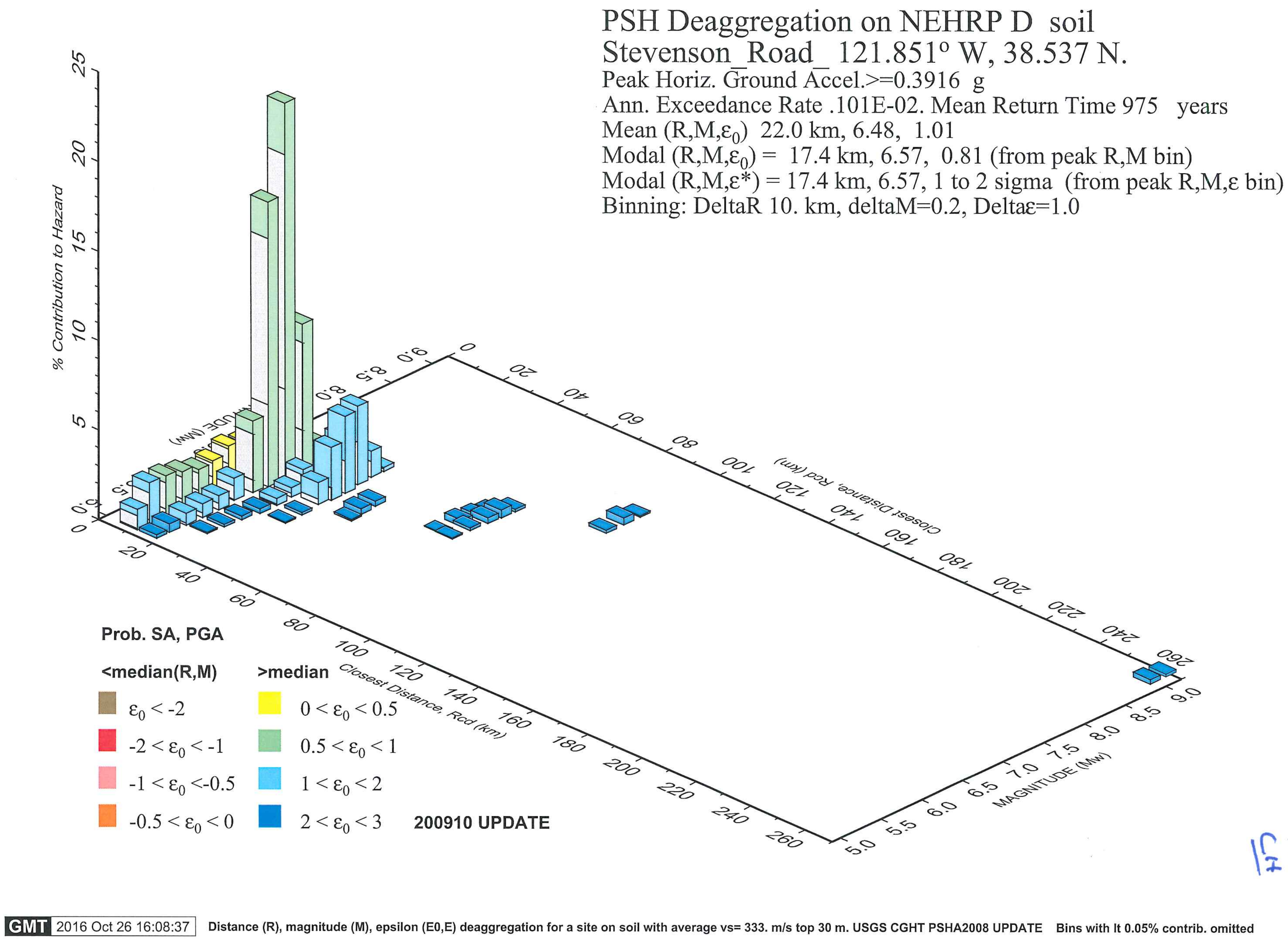
Point ID	Depth (ft)	Field SPT Blows (N)	Depth (m)	E_m	C_B	C_S	C_R	N_{60}	Approx. Unit Weight** (pcf)	Effective Overburder Stress (psf)	$N_{1,60}$	Soil Type	Sands* (SN) (m/s)	Silts* (SL) (m/s)	Cohesive* (C) (m/s)	Layer Thickness (m)	$V_{s\ layer}$ (m/s)	$V_{s\ 30}$ (m/s)
K_B-2	1.0	9	0.30	0.73	1.00	1.00	0.75	8	122	122	33	SL	-		-			
K_B-2	5.0	2	1.52	0.73	1.00	1.00	0.75	2	92	490	4	SL	-	211	-	4.6	238	
K_B-2	10.0	4	3.05	0.73	1.00	1.00	0.75	3	90	940	5	SL	-	264	-			
K_B-2	15.0	16	4.57	0.73	1.00	1.00	0.85	17	**132**	1600	19	C	-	-	310	1.5	310	
K_B-2	20.0	8	6.10	0.73	1.00	1.00	0.95	9	110	2150	9	SL	-	380	-	1.5	380	
K_B-2	25.0	11	7.62	0.73	1.00	1.00	0.95	13	128	2790	11	C	-	-	310			
K_B-2	30.0	20	9.14	0.73	1.00	1.00	1.00	24	135	3465	18	C	-	-	310			
K_B-2	35.0	17	10.67	0.73	1.00	1.00	1.00	21	130	4115	15	C	-	-	310			
K_B-2	40.0	10	12.19	0.73	1.00	1.00	1.00	12	**134**	4785	8	C	-	-	310			
K_B-2	45.0	9	13.72	0.73	1.00	1.00	1.00	11	130	5435	7	C	-	-	310	13.7	310	
K_B-2	50.0	9	15.24	0.73	1.00	1.00	1.00	11	128	6075	6	C	-	-	310			**307**
K_B-2	55.0	19	16.76	0.73	1.00	1.00	1.00	23	**130**	6413	13	C	-	-	310			
K_B-2	60.0	6	18.29	0.73	1.00	1.00	1.00	7	124	6721	4	C	-	-	310			
K_B-2	65.0	30	19.81	0.73	1.00	1.00	1.00	37	135	7084	19	C	-	-	310			
K_B-2	70.0	37	21.34	0.73	1.00	1.00	1.00	45	125	7397	23	SN	380	-	-	3.0	380	
K_B-2	75.0	18	22.86	0.73	1.00	1.00	1.00	22	120	7685	11	SN	380	-	-			
K_B-2	80.0	8	24.38	0.73	1.00	1.00	1.00	9	**122**	7983	5	C	-	-	310	1.5	310	
K_B-2	85.0	16	25.91	0.73	1.00	1.00	1.00	19	115	8246	9	SL	-	380	-	1.5	380	
K_B-2	90.0	11	27.43	0.73	1.00	1.00	1.00	14	125	8559	7	C	-	-	310			
K_B-2	95.0	28	28.96	0.73	1.00	1.00	1.00	34	130	8897	16	C	-	-	310	3.0	310	
K_B-2	100.0	41	30.48	0.73	1.00	1.00	1.00	50	135	9260	23	C	-	-	310			

* Shear wave velocity correlations from Appendix in Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations" November 2012. Based on 2010 UCLA study by S. Brandenberg.

** Approximate unit weights determined by SPT correlations in Caltrans Geotechnical Manual March 2014. Values in **BOLD** are measured densities


PSH Deaggregation on NEHRP D soil
Stevenson_Road_ 121.851° W, 38.537 N.
Peak Horiz. Ground Accel.>=0.3916 g
Ann. Exceedance Rate .101E-02. Mean Return Time 975 years
Mean (R,M,ε_0) 22.0 km, 6.48, 1.01
Modal (R,M,ε_0) = 17.4 km, 6.57, 0.81 (from peak R,M bin)
Modal (R,M,ε*) = 17.4 km, 6.57, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
$\varepsilon_0 < -2$
$-2 < \varepsilon_0 < -1$
$-1 < \varepsilon_0 < -0.5$
$-0.5 < \varepsilon_0 < 0$
$0 < \varepsilon_0 < 0.5$
$0.5 < \varepsilon_0 < 1$
$1 < \varepsilon_0 < 2$
$2 < \varepsilon_0 < 3$
200910 UPDATE
GMT 2016 Oct 26 16:08:37
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 333. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations

November 2012

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

Introduction

The Geotechnical Manual presents the requirements for determining and reporting seismic information and design response spectrum. This document presents the methodologies used to develop the deterministic, probabilistic and controlling design response spectra using the web tool ARS Online. Much of the information provided herein is based on the standards of the Caltrans Seismic Design Criteria Appendix B (SDC-B).

Section 1: Definitions for Developing Deterministic Acceleration Response Spectrum (ARS)

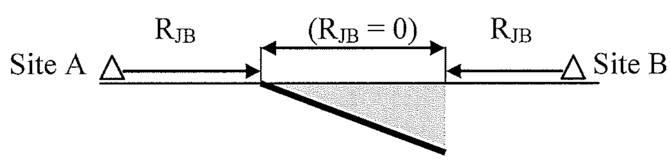
<u>Fault Parameters from the *Caltrans Fault Database*</u>

MMax	Maximum moment magnitude of fault - the largest earthquake a fault is capable of generating.
F_{RV}	Faults identified as a Reverse Fault "R" in the *Caltrans Fault Database*.
F_{NM}	Faults identified as a Normal Fault "N" in the *Caltrans Fault Database.*
δ	Fault dip angle (deg)
Z_{TOR}	Depth to top of rupture (km)
Z_{BOT}	Depth to bottom of rupture (km)

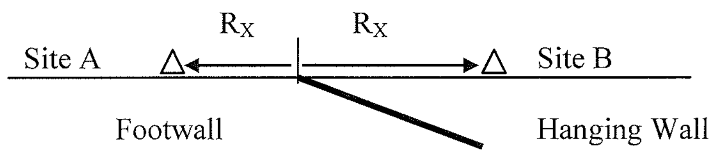
<u>Distance Terms</u>

R_{RUP} Closest distance (km) to the fault rupture plane, as shown in Appendix B.

R_{JB} Joyner-Boore distance - The shortest horizontal distance (km) to the surface projection of the rupture area. Think of this as the nearest horizontal distance to the area directly overlying the fault. R_{JB} is zero if the site is located within that area.



R_X Horizontal distance (km) to the fault trace or surface projection of the top of rupture plane. It is measured perpendicular to the fault (or the fictitious extension of the fault). The diagram below shows when a site is located on the footwall side (Site A) or on hanging wall side (Site B).



Appendix A: Determination of Shear Wave Velocity and V_{S30}

The average shear wave velocity (V_{S30}) for the upper 30 meters (100 feet) of the soil/rock profile is required to determine the design ground motion at the ground surface of a project site using the attenuation relationships included in the SDC. The equation used for calculating V_{S30} within the upper 30m (100 ft) is:

$$V_{S30} = \frac{100 \text{ ft}}{\frac{D_1}{V_1} + \frac{D_2}{V_2} + \frac{D_3}{V_3} + \ldots + \frac{D_n}{V_n}};$$

where D is the layer thickness (ft) and V is the shear wave velocity (ft/s) for that layer. **Note: V_{S30} input for attenuation models must be converted into meters/sec**.

The shear wave velocity (Vs) of each soil or rock layer may be measured in-situ or, where applicable, estimated based on empirical correlations with other parameters (e.g. field or lab data). In-situ measurements of V_s using geophysical methods, where feasible, are relatively simple and preferred for the purpose of estimating ground motions. The table below provides brief descriptions of some common geophysical methods for measuring V_s.

Geophysical Test	Brief Description
PS Suspension logging	Shear wave measurements are made in an open or thermoplastic-cased borehole. Source and receivers have a fixed separation and are both within the borehole. Shear wave velocities are measured at discrete depth intervals.
Down-hole seismic	The seismic source is fixed at the surface, and shear wave measurements are made with the receiver placed in the borehole at discrete intervals. Source to receiver separation varies with the receiver location.
Seismic CPT cone	The CPT seismic cone method is similar to the down-hole seismic method. The source is at the surface, the receiver is in the CPT cone. No predrilled borehole is necessary. Measurements are at discrete intervals.
Rayleigh Wave Inversion	Measurements are made at the surface. Several geophysical methods are available, including Refraction Microtremor (ReMi), Multichannel Analysis of Surface Waves (MASW) and Spectral Analysis of Surface Waves (SASW).

Note: If geophysical methods are used, the Geophysics Branch or a Professional Geophysicist should provide the data and the results of the test in a report.

In-situ measurements of V_s are generally required for project sites underlain by profile types that fall within the Type F category (as listed in SDC, Figure B.12), and in many cases Type E, or when site-specific dynamic ground response analyses are performed. For these cases, V_s measurements may need to extend to depths greater than 30 m depending on the site conditions and the appropriate depth of the input design base motion.

Shear Wave Velocity Correlations

In the absence of in-situ measurements, V_s for most soil layers and soft sedimentary rock layers can be estimated using established correlations. It should be noted that the correlations may contain significant uncertainty and geophysical measurements should be used when feasible. For cohesive soils, empirical correlations with laboratory measured undrained shear strength are preferred. For cohesionless soils, correlations with either the SPT (Standard Penetration Test – ASTM D1586) blow count value, N_{60} (blow count corrected for hammer efficiency but not for overburden), or the CPT tip resistance, q_t, may be used. Recommended shear wave velocity correlations for soil layers for use in the development of ground motions for State projects are presented below. A correlation with SPT blow counts is also provided for estimating Vs for young sedimentary rocks.

For stronger rock, the shear wave velocity for distinct zones may be evaluated based on correlations with other engineering and physical properties of rock mass and rock cores measured in the field or laboratory, or estimated by experienced geo-professionals using geologic correlations. Such properties include, but are not limited to, uniaxial compressive strength of rock, RQD of rock mass, elastic modulus, poisson's ratio and ultrasonic shear velocity (see Mayne et al, 2001).

Well documented and established empirical V_{S30} correlations specific to the earth material under consideration, or to the project site or the general area with similar earth material may be used by experienced geo-professionals provided adequate justifications of their use and the pertinent references are included in the report.

In 2007, UC Davis (DeJong 2007) compiled published correlations between shear wave velocity and common in-situ geotechnical test parameters and presented recommended correlations for various soil types. In 2010, a research study done by UC Los Angeles (Brandenberg, Bellana and Shantz, 2010) was published that reviewed Caltrans P-S Log data and SPT data from 79 borings and presented recommended correlations for various soil types. Both studies were reviewed and compared by Caltrans for consistency with the ATC-32 profile types (SDC, Figure B.12) and the parameters and ranges used to define the profile types. The correlations provided below are recommended for State project sites, where applicable.

The geo-professionals should be aware of the limitations of each correlation used. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data that are reliable and representative of the actual site conditions. If correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available in a region, then in-situ measurements are recommended.

The correlation equations below provide shear wave velocity in m/sec. and are valid for a range of 100 to 380 m/sec for cohesionless soil, and are valid for a range of 100 to 310 m/sec for cohesive soil. Higher velocities should be verified with in-situ measurements. Also, shear wave velocity correlations should not be applied to depths less than 5ft. Estimation of shear wave velocities (V_S) at depths less than 5ft can be approximated by a V_S value at a depth of 5ft.

- Cohesionless Soil

The shear wave velocity of cohesionless soil may be estimated by using correlations with SPT blow counts or CPT tip resistance. Correlations for cohesionless soil using SPT data and effective overburden stress developed in the 2010 UCLA study are:

$$V_S = \exp(4.045 + 0.096(\ln(N_{60})) + 0.236(\ln(\sigma'_v)) \quad \text{For Sand}$$

$$V_S = \exp(3.783 + 0.178(\ln(N_{60})) + 0.231(\ln(\sigma'_v)) \quad \text{For Silt}$$

Where N_{60} is the SPT blow count corrected only for the hammer energy and σ'_v is the effective overburden stress (kPa). These correlations are valid where $N_{60} \leq 100$ and $\sigma'_v \leq 506$ kPa.

For CPT data, the correlation adapted by Mayne (2007) after Baldi et al, (1989) is recommended to calculate shear wave velocity for cohesionless soil:

$$V_S = 277\left(q_t\right)^{0.13}\left(\sigma'_{vo}\right)^{0.27}$$

Where q_t and σ'_{vo} are the CPT tip resistance (MPa) and effective overburden stresses (MPa).

- Cohesive Soil

The correlation by Dickenson (1994) using undrained shear strength to calculate shear wave velocity is recommended for cohesive soil:

$$V_S = 203\left(S_u / p_a\right)^{0.475}$$

Where, S_u is the undrained shear strength of cohesive soil and p_a is the atmospheric pressure in the same unit as S_u.

In the absence of undrained shear strength data, the shear wave velocity of cohesive soil may be estimated by using the CPT correlation developed by Mayne and Rix (1995):

$V_S = 1.75\left(q_t\right)^{0.627}$ where q_t is the measured CPT tip resistance (kPa).

When undrained shear strength or CPT tip resistance data are not available, use the correlation using SPT data and effective overburden developed in the 2010 UCLA study (where $N_{60} \geq 3$):

$V_S = \exp(3.996 + 0.230(\ln(N_{60})) + 0.164(\ln(\sigma'_v))$ For Cohesive Soil

- Young Sedimentary Rock

Imai and Tonouchi (1982) reviewed over a hundred SPTs with corresponding V_s in young sedimentary rocks (Tertiary deposits). For these types of rock, their "Tertiary Sand/Clay" correlation may be used to estimate shear wave velocity.

$$V_S = 109\left(N_{60}\right)^{0.319}$$

The V_s value estimated using the SPT correlation for young sedimentary rock layers should be limited to 560 m/sec.

Other Rocks

While there are numerous studies that correlate shear wave velocity to in-situ geotechnical testing of soil, there are relatively few studies that correlate shear wave velocity to physical properties of rock.

Two notable studies that may be useful to geo-professionals in developing approximations of shear wave velocities based on physical properties of rock are provided below. Fumal (1978) correlated shear wave velocity to weathering, hardness, fracture spacing, and lithology based on data from 27 sites in the upland areas of the San Francisco (Bay Area) region. Fumal and Tinsley (1985) extended the 1978 study to include 84 sites in the Los Angeles region. Some physical properties of the rock were more important than others depending on lithology, soil texture, rock hardness, but fracture spacing was found to be the most important factor affecting shear wave velocity. A thorough review of these studies will significantly aid geo-professionals in their estimations of shear velocity.

In the absence of in-situ measurements of Vs, the V_{S30} for competent rock in California should be limited to 760 m/sec.

- Estimating V_{S30} for sites with subsurface information less than 100 ft (30 m)

For borings shallower than 100 feet (30 meters) are not available, V_{S30} can be determined by extrapolating shallower Vs data assuming that no significant changes in the subsurface occur to the extrapolated depth of 100 feet (David Boore (2004)):

$V_{S30} = [1.45 - (0.015 * d)] * V_{s(d)}$; where d is the depth in meters to the bottom of the known soil column and Vs(d) is the average shear wave velocity (m/sec) for a known depth.

CE&G
CAL ENGINEERING & GEOLOGY

DOCUMENT REVIEW COVER SHEET

1. PROJECT NAME Stevenson Road Bridge Design	2. PROJECT NUMBER 160600
3. DOCUMENT TITLE Passive And Friction Resistance Based On 2005 Kleinfelder Borings B-1 And B-2	
4. DOCUMENT STATUS DESIGNATION ☐ Preliminary ☒ Final ☐ Cancelled	
5. NOTES/COMMENTS The purpose of this calculation is to provide recommendations on abutment wall passive resistance and abutment footing frictional resistance based solely on Kleinfelder's borings.	

ATTACHMENTS	TOTAL NO. OF PAGES
Hand Calculations	3
Kleinfelder Boring Logs, N60 Blow Count Calculations	7
caltrans Geotechncial Manual Soil Correlations	5
Friction Angle Correlation And Navfac Passive Coefficient	2

RECORD OF REVISIONS

6. No.	7. REASON FOR REVISION	8. TOT. PGS	10. ORIGINATOR (PRINT/SIGN/DATE)	11. CHECKER (PRINT/SIGN/DATE)	12. QA/QC (PRINT/SIGN/DATE)	13. APPRVD./ACCPTD (PRINT/SIGN)	14. DATE (M/D/YY)
1	Initial Issue	16	Mehal Vitthal 7-7-16	Chris Hockett 7/7/16	Mark Myers 7/7/16	Phil Gregory	7/7/16 7/7/16

M.V
CH

SLIDING RESISTANCE (PASSIVE & FRICTIONAL RESISTANCE)

REF: AASHTO LRFD BRIDGE DESIGN SPECS (AASHTO LRFD BDS) 2012

$$R_R = \phi R_n = \phi_\tau R_\tau + \phi_{ep} R_{ep}$$ AASHTO LRFD BDS (10.6.3.4-1)

where: ϕ_τ = RESISTANCE FACTOR FOR SHEAR BETWEEN SOIL & FOUNDATION

ϕ_{ep} = RESISTANCE FACTOR FOR PASSIVE RESISTANCE

R_τ = NOMINAL SHEAR RESISTANCE BETWEEN SOIL & FOUNDATION

R_{ep} = NOMINAL PASSIVE RESISTANCE

$\phi_\tau = 0.80$
$\phi_{ep} = 0.50$
FROM TABLE 10.5.5.2.2-1 AASHTO LRFD BDS

ASSUME
CAST-IN-PLACE CONCRETE

$$R_\tau = V \tan(\phi_f)$$

ASSUMING FOOTING IS FOUNDED ON COHESIONLESS SOIL
ASSHTO LRFD BDS (10.6.3.4-2)

where: V = total vertical FORCE

ϕ_f = INTERNAL FRICTION ANGLE OF DRAINED SOIL

ASSUMING THE DESIGN METHOD OUTLINED ABOVE IS UTILIZED:

(1) SOIL INTERNAL FRICTION ANGLE

BASED ON EXISTING BORINGS AT ABUTMENTS $\rightarrow N_{1,60} = 4$ AT DEPTH NEAR FOOTING BOTTOM

REF: CALTRANS GEOTECHNICAL MANUAL (ATTACHED)

CHART 1: SOIL CORRELATIONS (AFTER BOWLES, 1977)

FOR $N_{1,60} = 4 \rightarrow \phi_f = 28°$

$\tan(\phi_f) = \tan(28°) = 0.53$ ← FRICTION FACTOR

CROSS REFERENCE WITH CIVIL ENGR. REFERENCE MANUAL (ATTACHED)

SILTY SANDS

M.V
CH

CE&G CAL ENGINEERING & GEOLOGY

Project: STEVENSON RD BRIDGE | Project No.: 160600 | Sheet: 2 of 3
Item: PASSIVE & FRICTION RESISTANCE - KLEINFELDER BORINGS | Designer: MEHAL V. | Date: 07/06/16
Checker: | Date:

② SOIL UNIT WEIGHT

MEASURED VALUES THROUGH FIRST 15 FEET

BASED ON EXISTING BORINGS AT ABUTMENTS } → $N_{1,60} = 9 ; 3 ; 4$

REF: CALTRANS GEOTECHNICAL MANUAL (ATTACHED)

CHART 2: SOIL CORRELATIONS (AFTER BOWLES, 1977)

$N_{1,60} = 9 \rightarrow \gamma = 108$ PCF
$N_{1,60} = 3 \rightarrow \gamma = 84$ PCF
$N_{1,60} = 4 \rightarrow \gamma = 90$ PCF

} AVERAGE THESE VALUES → $\gamma = 94$ PCF

AVG. MOIST UNIT WEIGHT OF BACKFILL MATERIAL BASED ON SPT BLOW COUNT OF EXISTING BORINGS

③ PASSIVE PRESSURES

REF: NAVFAC DESIGN MANUAL 7.02

FIGURE 3: ACTIVE AND PASSIVE COEFFICIENTS, SLOPING BACKFILL (GRANULAR SOILS)

LONGITUDINAL DIRECTION (RESISTANCE FROM ABUTMENT BACKWALL)

LEVEL ROAD BEHIND ABUTMENT

Given $\beta = 0^\circ \rightarrow K_p = 2.8$ (COULOMB THEORY)

TRANSVERSE DIRECTION (RESISTANCE FROM ABUTMENT WINGWALLS)

Given $\beta = 0^\circ \rightarrow K_p = 2.8$ (COULOMB THEORY)

ASSUME LEVEL EMBANKMENT

EQUIVALENT FLUID PRESSURE (BOTH LONGITUDINAL & TRANSVERSE)

$EFP = \gamma * K_p = 94 \text{ PCF} * 2.8$

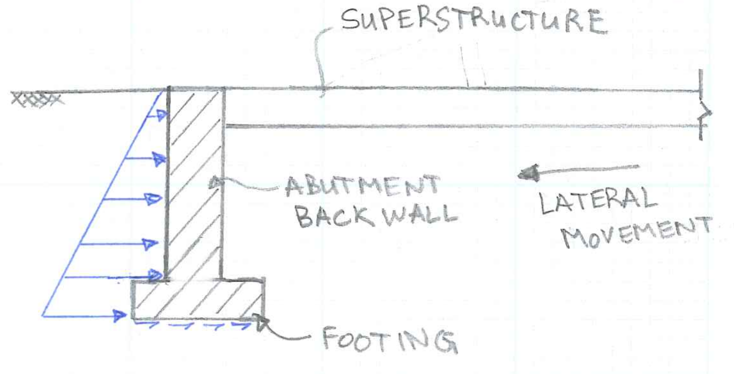
EFP = 263 PSF PER FT OF DEPTH

M.V
C.A.

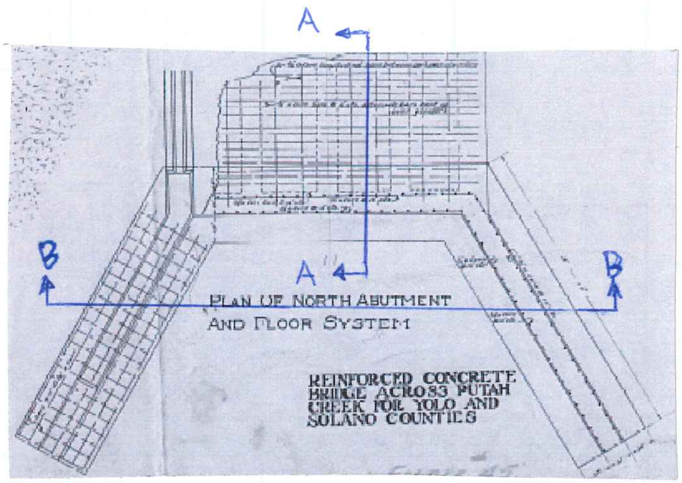
LONGITUDINAL DIRECTION

- EFP = 263 $\frac{\text{PSF}}{\text{FT OF DEPTH}}$
- PASSIVE RESISTANCE ONLY MOBILIZED BY SOIL MASS RETAINED BY ABUTMENT BACKWALL
- $\tan(\phi_F) = 0.53$

  $\phi_F = 28°$



LONGITUDINAL RESISTANCE (A-A)

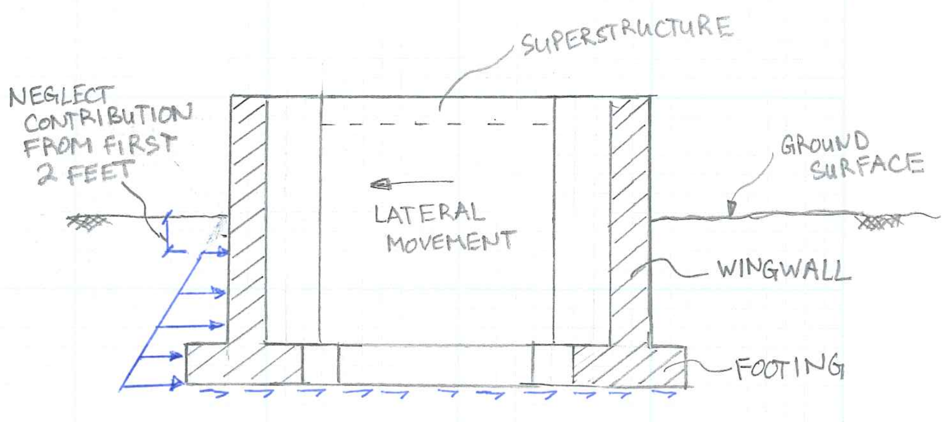


PLAN VIEW

TRANSVERSE DIRECTION

- EFP = 263 $\frac{\text{PSF}}{\text{FT OF DEPTH}}$
- PASSIVE RESISTANCE ONLY MOBILIZED BY SOIL MASS ON OUTER SIDES OF WINGWALLS.
- NEGLECT THE PASSIVE CONTRIBUTION FROM THE FIRST TWO (2) FEET OF SOIL
- $\tan(\phi_F) = 0.53$

  $\phi_F = 28°$



TRANSVERSE RESISTANCE (B-B)

M.V

CH

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (Half or more of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (Half or more of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit equal to or greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
VARIABLEY WEATHERED BEDROCK				SANDY SILTSTONE
				SILTSTONE
				SILTSTONE - CLAYSTONE
				CLAYSTONE
				SANDSTONE

LOG KEY SYMBOLS

- BULK / BAG SAMPLE
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)
- MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)
- SHELBY TUBE (3 inch outside diameter)
- CALIFORNIA SAMPLER (3 inch outside diameter)
- NO RECOVERY
- WATER LEVEL (level after completion)
- WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKLY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SV	PARTICLE SIZE ANALYSIS
PI	PLASTICITY INDEX	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SS	SOLUBLE SULFATES
P	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
SF	SOIL FERTILITY		

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16" - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2" - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATE A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

KLEINFELDER

Date: 12-1-05
Drawn by: J. Gilbert
Project Number: 63601
Filename: USCS/Log Key

UNIFIED SOIL CLASSIFICATION SYSTEM / LOG KEY

STEVENSON BRIDGE
YOLO/SOLANO COUNTIES, CALIFORNIA

PLATE A-1

M.V
CH

Surface Conditions:	Asphalt Road	Date Completed:	12/28/2005
Groundwater:	Groundwater encountered at a depth of approximately 50 feet below existing site grade during drilling.	Logged By:	P. Sorci
		Total Depth:	Approximately 101-1/2 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 93 feet (msl)

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
	0													Asphalt Concrete: approximately 3"
														SILT with trace of Sand (ML): brown, moist, stiff to very stiff, fine sand
90			B2-1-B B2-1-A	15										
	5													Sandy SILT (ML): brown, moist, soft to medium stiff
			B2-2-B B2-2-A	4										
85														SILT with Sand (ML): brown, dry to moist, medium stiff
	10		B2-3-B B2-3-A	6										
80														Fat CLAY (CH): dark brown, moist, very stiff to hard, high plasticity
	15		B2-4-B B2-4-A	27		114	16					UC=21,953 psf @ 5% Strain		
75														SILT (ML): brown, dry to moist, stiff to very stiff
	20		B2-5-B B2-5-A	13										
70														Lean CLAY (CL): brown mottled olive-brown, dry to moist, very stiff, some plasticity
	25		B2-6-B B2-6-A	19										
65														

Blow count used

APPROX. BOT OF FOOTING

KLEINFELDER

LOG OF BORING B-2 SOUTH ABUTMENT

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
1 of 4
A-3

Drafted By: J. Gilbert
Date: 4/27/2006
Project No.: 63601-1
File Number: stevenson bridge

M.V
CH

Surface Conditions:	Asphalt Road	Date Completed:	12/27/2005
Groundwater:	Groundwater encountered at a depth of approximately 46-1/2 feet below existing site grade during drilling.	Logged By:	P. Sorci
		Total Depth:	Approximately 101-1/2 feet
Method:	Hollow Stem Auger	Boring Diameter:	8 inch
Equipment:	BK-57 Truck Mounted Drill Rig		

Approximate Elevation: 94 feet (msl)

Elevation (ft., msl)	Depth (feet)	Sample Type	Sample No.	Blows/ft	Pocket Penetrometer (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests	Lithography	DESCRIPTION
														Asphalt Concrete: approximately 5"
														SILT (ML): brown, dry, hard, trace of fine sand
			B1-1-B B1-1-A	7								Corrosion: see Appendix		- grades medium stiff
90	5		B1-2-B B1-2-A	41	2.75							Corrosion: see Appendix		- grades with trace of Clay, very stiff
85	10		B1-3-B B1-3-A	33										- grades with fine Sand, very stiff to hard
80	15		B1-4-B B1-4-A	44	2.5	89	13					UC=350 psf @ 2% Strain		- grades with some fine Sand, hard, no cohesion
														Poorly Graded Silty SAND (SP-SM): brown, dry to moist, loose, fine sand, with fines
75	20		B1-5-B B1-5-A	9							29			
70	25		B1-6-B B1-6-A	54										- grades moist, dense

APPROX. BOT OF FOOTING

SAC 2004 63601 STEVENSON BRIDGE.GPJ 4/7/06

KLEINFELDER

Drafted By: J. Gilbert
Date: 3/21/2006

Project No.: 63601-1
File Number: stevenson bridge

LOG OF BORING B-1 NORTH ABUTMENT

STEVENSON BRIDGE
YOLO/SOLANO COUNTY, CALIFORNIA

PLATE
1 of 4
A-2

ELEVATION VS. N60 BLOW COUNT

Kleinfelder Broing Data											DATA ANALYSIS									
														Correction Factors						
Point ID	Depth (feet)	Approx. Surface Elev (feet,MSL)	Length (inch)	Type	Raw Blow Count (N)	Pocket Penetrometer (tsf)	Liquid Limit	Plasticity Index	Pass #4 sieve (%)	Pass #200 sieve (%)	Hammer Weight (lb)	Blow Elev (feet,MSL)	Blow Count (N)	Hammer (1.0 for 140 lb, 0.5 for 70 lb)	CM to SPT (0.6)	E_m	C_B	C_S	C_R	N_{60}
B-1	1.0	94	18	MCAL	7	-	-	-	-	-	140	93	7	1	4	0.73	1.15	1.0	0.75	**4**
B-1	5.0	94	18	MCAL	41	2.75	-	-	-	-	140	89	41	1	25	0.73	1.15	1.0	0.75	**26**
B-1	10.0	94	18	MCAL	33	-	-	-	-	-	140	84	33	1	20	0.73	1.15	1.0	0.75	**21**
B-1	15.0	94	18	MCAL	44	2.5	-	-	-	-	140	79	44	1	26	0.73	1.15	1.0	0.85	**31**
B-1	20.0	94	18	MCAL	9	-	-	-	-	29	140	74	9	1	5	0.73	1.15	1.0	0.95	**7**
B-1	25.0	94	18	MCAL	54	-	-	-	-	-	140	69	54	1	32	0.73	1.15	1.0	0.95	**43**
B-1	30.0	94	18	MCAL	13	-	-	-	-	-	140	64	13	1	8	0.73	1.15	1.0	1.00	**11**
B-1	35.0	94	18	MCAL	60	-	-	-	-	-	140	59	60	1	36	0.73	1.15	1.0	1.00	**50**
B-1	40.0	94	18	MCAL	83	-	-	-	-	-	140	54	83	1	50	0.73	1.15	1.0	1.00	**70**
B-1	45.0	94	18	MCAL	35	-	-	-	-	-	140	49	35	1	21	0.73	1.15	1.0	1.00	**29**
B-1	50.0	94	18	MCAL	11	1.75	-	-	-	-	140	44	11	1	7	0.73	1.15	1.0	1.00	**9**
B-1	55.0	94	18	MCAL	20	2.0	40	21	-	-	140	39	20	1	12	0.73	1.15	1.0	1.00	**17**
B-1	60.0	94	18	MCAL	12	1	-	-	-	-	140	34	12	1	7	0.73	1.15	1.0	1.00	**10**
B-1	65.0	94	18	MCAL	23	-	-	-	-	-	140	29	23	1	14	0.73	1.15	1.0	1.00	**19**
B-1	70.0	94	18	MCAL	61	-	-	-	100	6	140	24	61	1	37	0.73	1.15	1.0	1.00	**51**
B-1	75.0	94	18	MCAL	79	-	-	-	-	-	140	19	79	1	47	0.73	1.15	1.0	1.00	**66**
B-1	80.0	94	18	MCAL	20	-	-	-	-	-	140	14	20	1	12	0.73	1.15	1.0	1.00	**17**
B-1	85.0	94	18	MCAL	34	-	-	-	-	-	140	9	34	1	20	0.73	1.15	1.0	1.00	**29**
B-1	90.0	94	18	MCAL	34	-	-	-	-	-	140	4	34	1	20	0.73	1.15	1.0	1.00	**29**
B-1	95.0	94	18	MCAL	51	-	-	-	-	-	140	-1	51	1	31	0.73	1.15	1.0	1.00	**43**
B-1	100.0	94	18	MCAL	69	-	-	-	-	-	140	-6	69	1	41	0.73	1.15	1.0	1.00	**58**

Kleinfelder Boring Data											DATA ANALYSIS									
														Correction Factors						
Point ID	Depth (feet)	Approx. Surface Elev (feet,MSL)	Length (inch)	Type	Raw Blow Count (N)	Pocket Penetrometer (tsf)	Liquid Limit	Plasticity Index	Pass #4 sieve (%)	Pass #200 sieve (%)	Hammer Weight (lb)	Blow Elev (feet,MSL)	Blow Count (N)	Hammer (1.0 for 140 lb, 0.5 for 70 lb)	CM to SPT (0.6)	E_m	C_B	C_S	C_R	N_{60}
B-2	1.0	93	18	MCAL	15	-	-	-	-	-	140	92	15	1	9	0.73	1.15	1.0	0.75	**9**
B-2	5.0	93	18	MCAL	4	-	-	-	-	-	140	88	4	1	2	0.73	1.15	1.0	0.75	**3**
B-2	10.0	93	18	MCAL	6	-	-	-	-	-	140	83	6	1	4	0.73	1.15	1.0	0.75	**4**
B-2	15.0	93	18	MCAL	27	-	-	-	-	-	140	78	27	1	16	0.73	1.15	1.0	0.85	**19**
B-2	20.0	93	18	MCAL	13	-	-	-	-	-	140	73	13	1	8	0.73	1.15	1.0	0.95	**10**
B-2	25.0	93	18	MCAL	19	-	-	-	-	-	140	68	19	1	11	0.73	1.15	1.0	0.95	**15**
B-2	30.0	93	18	MCAL	33	-	-	-	-	-	140	63	33	1	20	0.73	1.15	1.0	1.0	**28**
B-2	35.0	93	18	MCAL	29	-	-	-	-	-	140	58	29	1	17	0.73	1.15	1.0	1.0	**24**
B-2	40.0	93	18	MCAL	16	-	-	-	-	-	140	53	16	1	10	0.73	1.15	1.0	1.0	**13**
B-2	45.0	93	18	MCAL	15	-	-	-	-	-	140	48	15	1	9	0.73	1.15	1.0	1.0	**13**
B-2	50.0	93	18	MCAL	15	-	35	18	-	-	140	43	15	1	9	0.73	1.15	1.0	1.0	**13**
B-2	55.0	93	18	MCAL	32	-	-	-	-	-	140	38	32	1	19	0.73	1.15	1.0	1.0	**27**
B-2	60.0	93	18	MCAL	10	-	-	-	-	-	140	33	10	1	6	0.73	1.15	1.0	1.0	**8**
B-2	65.0	93	18	MCAL	50	-	-	-	-	-	140	28	50	1	30	0.73	1.15	1.0	1.0	**42**
B-2	70.0	93	18	MCAL	61	-	-	-	-	-	140	23	61	1	37	0.73	1.15	1.0	1.0	**51**
B-2	75.0	93	18	MCAL	30	-	-	-	63	3	140	18	30	1	18	0.73	1.15	1.0	1.0	**25**
B-2	80.0	93	18	MCAL	13	-	-	-	-	-	140	13	13	1	8	0.73	1.15	1.0	1.0	**11**
B-2	85.0	93	18	MCAL	26	-	-	-	-	-	140	8	26	1	16	0.73	1.15	1.0	1.0	**22**
B-2	90.0	93	18	MCAL	19	-	-	-	-	-	140	3	19	1	11	0.73	1.15	1.0	1.0	**16**
B-2	95.0	93	18	MCAL	47	-	-	-	-	-	140	-2	47	1	28	0.73	1.15	1.0	1.0	**39**
B-2	100.0	93	18	MCAL	68	-	-	-	-	-	140	-7	68	1	41	0.73	1.15	1.0	1.0	**57**

MW
CH

ELEVATION VS. DENSITY

Kleinfleder Borings					DATA ANALYSIS		
PointID	Depth (feet)	Approx. Surface Elev. (feet,MSL)	Dry Density (pcf)	Moisture Content (%)	Total Density (pcf)	Approx. Density Elev (feet,MSL)	Boyant Density (pcf)
B-1	15	94	89	13	**100.57**	79	38.2
B-1	50	94	102	25	**127.50**	44	65.1
B-1	80	94	90	33	**119.70**	14	57
B-2	15	93	114	16	**132.24**	78	70
B-2	40	93	114	18	**134.52**	53	72
B-2	55	93	105	23	**129.15**	38	67
B-2	80	93	93	31	**121.83**	13	59

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60}$$

where:
N_{60} = SPT N-value corrected for field procedures
E_m = hammer efficiency (from Table 3.3)
C_B = borehole diameter correction (from Table 3.4)
C_S = sampler correction (from Table 3.4)
C_R = rod length correction (from Table 3.4)
N = SPT N-value recorded in the field

*Equation and tables obtained from Coduto 1999: "Geotechnical Engineering - Principles and Practices"

TABLE 3.3 SPT HAMMER EFFICIENCIES (Adapted from Clayton, 1990).

Country	Hammer Type	Hammer Release Mechanism	Hammer Efficiency E_m
Argentina	Donut	Cathead	0.45
Brazil	Pin Weight	Hand Dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
	Donut	Cathead	0.50
Colombia	Donut	Cathead	0.50
Japan	Donut	Tombi trigger	0.78 - 0.85
	Donut	Cathead 2 turns + special release	0.65 - 0.67
UK	Automatic	Trip	0.73
USA	Safety	2 turns on cathead	0.55 - 0.60
	Donut	2 turns on cathead	0.45
Venezuela	Donut	Cathead	0.43

Description of Drilling

Drilling was completed using 140 lb automatic hammer. (see Table 3.3). The borehole diameter was 8 inches (see Table 3.4). The SPT samplers did not have liners and the Modified California samplers had liners. A value of 1.0 from Table 3.4 was selected. It was found that this did not have a significant affect on the N60 values.

TABLE 3.4 BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS (Adapted from Skempton, 1986).

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	65 - 115 mm (2.5 - 4.5 in)	1.00
	150 mm (6 in)	1.05
	200 mm (8 in)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner (not recommended)	1.20
Rod length factor, C_R	3 - 4 m (10 - 13 ft)	0.75
	4 - 6 m (13 - 20 ft)	0.85
	6 - 10 m (20 - 30 ft)	0.95
	> 10 m (> 30 ft)	1.00

CF
MC

M.V

CH

SOIL CORRELATIONS

Introduction

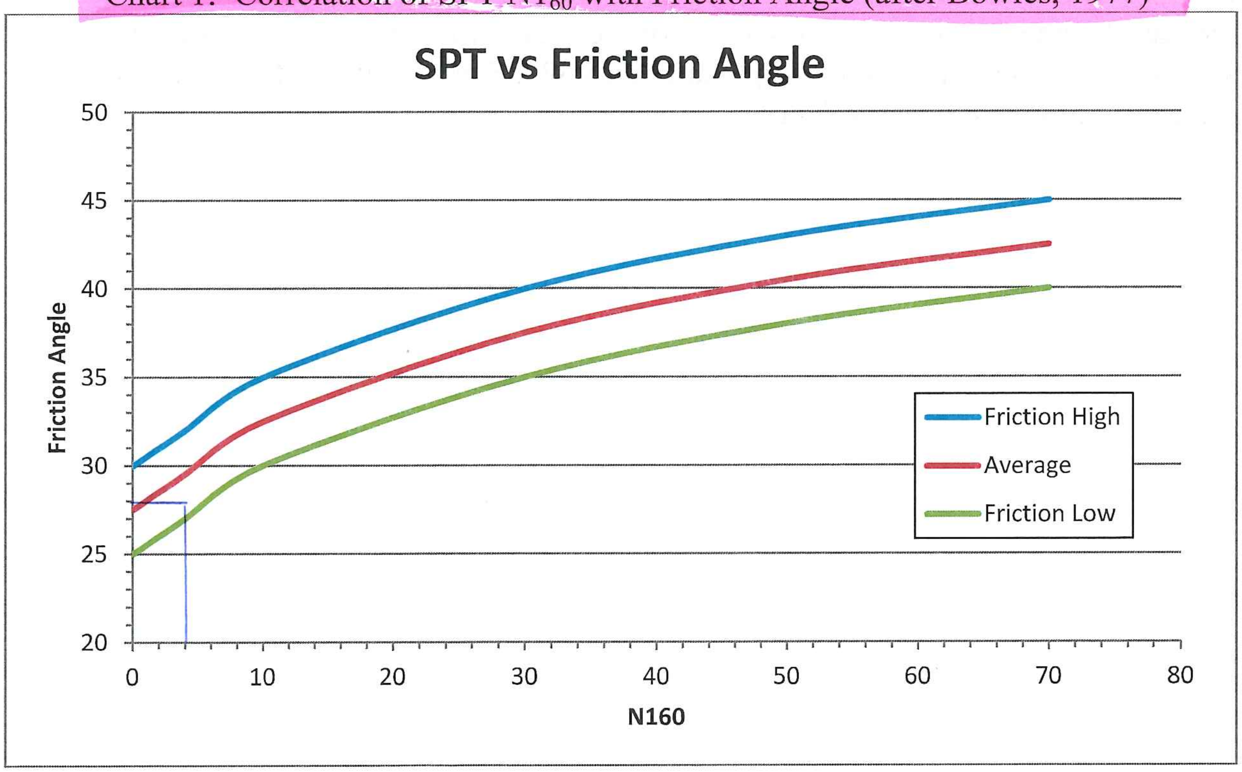
This section of the Geotechnical Manual presents the SPT correlations to be used for friction angle (phi angle) and unit weight. The correlations use Standard Penetration Test (N) values corrected for overburden and hammer efficiency ($N1_{60}$). Usage of correlations for geotechnical design is addressed in the various design sections of the Geotechnical Manual. Other correlations, e.g. CPT correlations and shear wave velocity correlations are found elsewhere in the Geotechnical Manual.

The correlations presented herein are after Bowles (1977), which is consistent with many of the NHI manuals used by the Department.

Granular Soil – Friction Angle

Use Chart 1 to correlate $N1_{60}$ to the friction (phi) angle.

Chart 1: Correlation of SPT $N1_{60}$ with Friction Angle (after Bowles, 1977)



Choose the friction angle (expressed to the nearest degree) based upon the soil type, particle size(s), and rounding or angularity. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant (about 30+ %) silt-sized material will fall in the lower portion of the range. Coarser materials with less than 5% fines will fall in the upper portion of the range. The extreme range of phi angles for any $N1_{60}$ is five degrees, so the adjustment factors for particle size and roundness should be only a degree or two. The following bullets provide help in determining which value to select for a given $N1_{60}$ and soil type:

- Use the maximum value for GW
- Use the average for GM and SP
- Use the minimum for SC
- Use the minimum + 0.5 for ML
- Use the average +1 for SW
- Use the average -1 for GC
- Use the Maximum -1 for GP

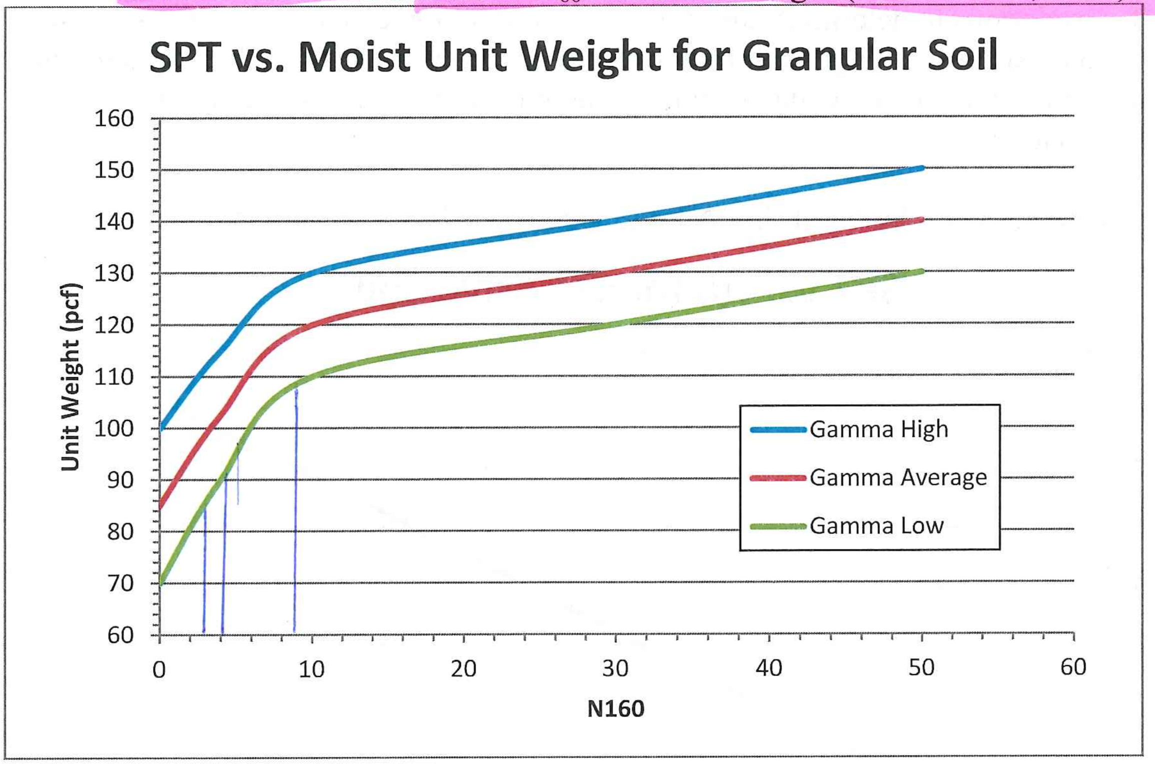
Values may also be increased with increasing grain size and/or particle angularity, and decreased with decreasing grain size and/or increasing roundness. For example, an SP with $N1_{60} =$ 30 could be assigned phi angles of 37, 38 or 39 degrees for fine, medium and coarse grain sizes respectively.

M.V
C.H.

Granular Soil - Unit Weight

Use Chart 2 to correlate $N1_{60}$ to the moist unit weight for granular soil.

Chart 2: Correlation of SPT $N1_{60}$ with Unit Weight (after Bowles, 1977).



Choose the unit weight expressed to the nearest five pcf for the soil type based on the following guidelines:

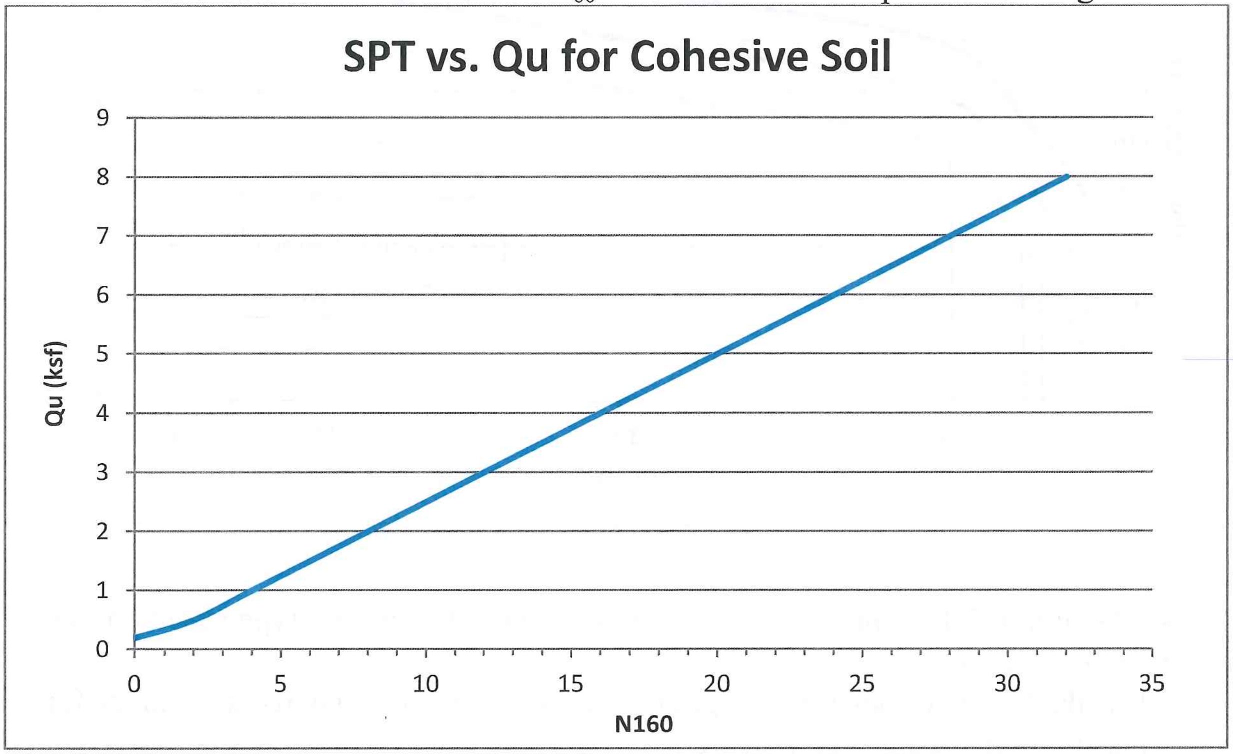
- Use the higher values for well-graded sands and gravels and average values for poorly-graded sands and gravels.
- Use lower values for elastic silt, and clayey or silty sands and gravel.
- Deduct up to 20% for dry soils.

M.V
CH

Cohesive Soil - Unconfined Compressive Strength (Qu)/Undrained Shear Strength (Su)

The standard practice is to determine shear strength of cohesive soils in the field based on measurements with torvane, pocket penetrometer, or vane shear. It is not acceptable to use SPT correlations to determine shear strength or to assign consistency values. Use Chart 3 to assign shear strength values when only SPT values are available. Usually this is applicable when data are available from old as-built LOTBs where field or laboratory strength tests are not available.

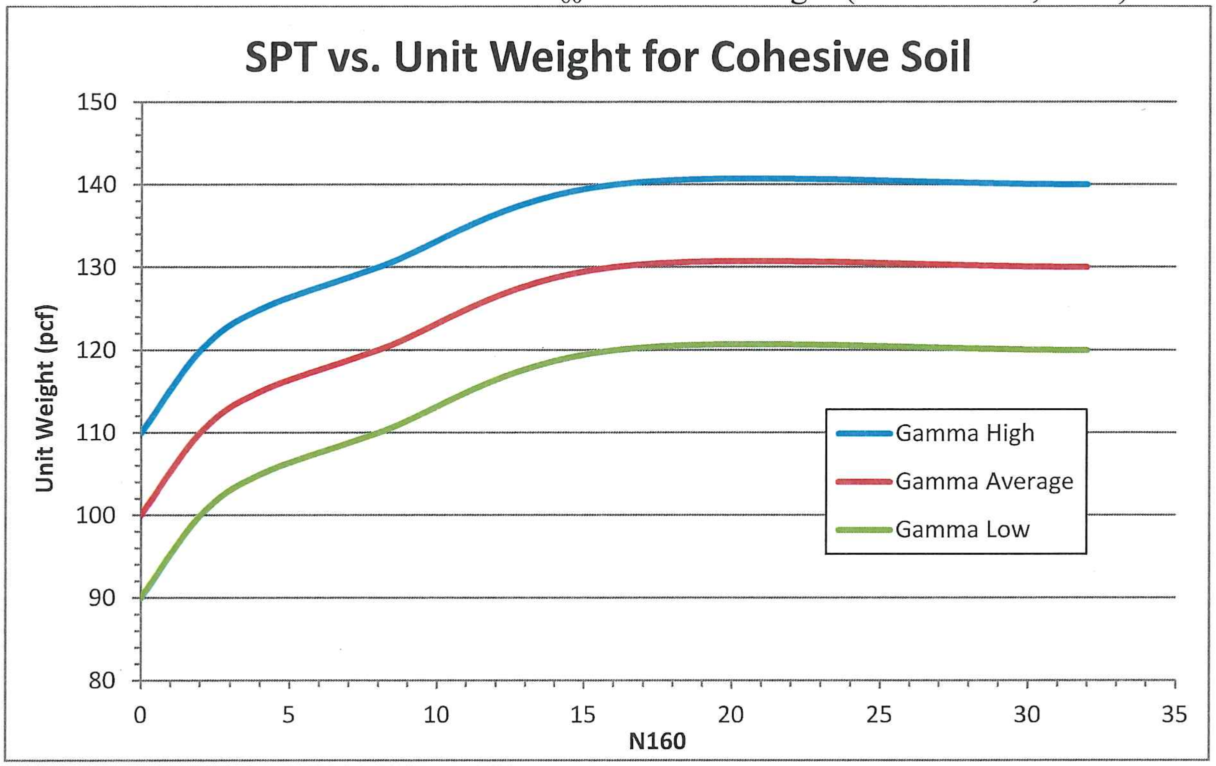
Chart 3: Correlation of SPT $N1_{60}$ to Unconfined Compressive Strength



Cohesive Soil - Unit Weight

Use Chart 4 to correlate $N1_{60}$ with the Unit Weight of cohesive soil.

Chart 4: Correlation of SPT $N1_{60}$ with Unit Weight (after Bowles, 1977).



Comparing field pocket penetrometer and/or torvane readings to Chart 4 is a good way of determining whether high or low values should be used. For example, if the pocket penetrometer reading for a clay with $N1_{60} = 10$ is about 2.5 ksf (the same as the value shown in Chart 3) the unit weight should correspond to the average value. If the pocket penetrometer reading is higher, the unit weight should be increased from the average, and if the pocket penetrometer reading is lower, the unit weight should be decreased from the average.

In the absence of SPT data, unit weights can be estimated using Charts 3 and 4 and the strength data (e.g., pocket penetrometer reading). For example, from Chart 3, a pocket penetrometer value of 5 ksf corresponds to an SPT $N1_{60}$ value of 20. Chart 4 shows the average unit weight of a cohesive soil with SPT $N1_{60} = 20$ is 130 pcf.

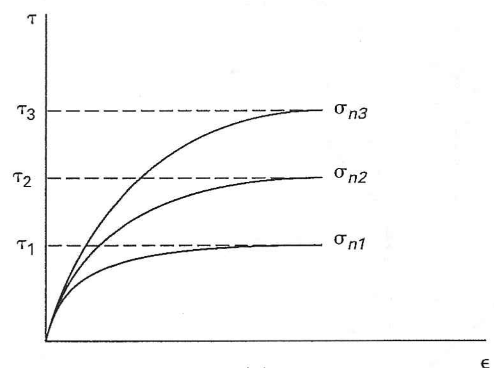
References

Bowles, J. E., 1977, *Foundation Analysis and Design*, McGraw-Hill, Inc., New York

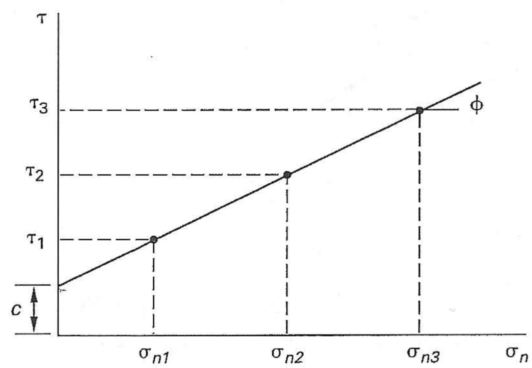
M.V
CH

into the direct shear box. The box has a top half and a bottom half that can slide laterally with respect to each other. A normal stress, σ_n, is applied vertically, and then one half of the box is moved laterally relative to the other at a constant rate. Measurements of vertical and horizontal displacement, δ, and horizontal shear load, P_h, are taken. The test is usually repeated at three different vertical normal stresses.

Because of the box configuration, failure is forced to occur on a horizontal plane. Results from each test are plotted as horizontal displacement versus horizontal stress, τ_h (horizontal force divided by the nominal area). *Failure* is determined as the maximum value of horizontal stress achieved. The vertical normal stress and failure stress from each test are then plotted in Mohr's circle space of normal stress versus shear stress.



(a)
stress-strain curves



(b)
Mohr's failure envelope

Figure 35.13 *Graphing Direct-Shear Test Results*

A line drawn through all of the test values is called the *failure envelope* (*failure line* or *rupture line*). The equation for the failure envelope is given by *Coulomb's equation*, which relates the strength of the soil, S, to the normal stress on the failure plane.[10,11,12]

$$S = \tau = c + \sigma \tan \phi \qquad 35.37$$

[10]Equation 35.37 is also known as the Mohr-Coulomb equation.
[11]The ultimate shear strength may be given the symbol S in some soils books.
[12]τ and σ in Coulomb's equation are the shear stress and normal stress, respectively, on the failure plane at failure.

ϕ is known as the *angle of internal friction*.[13] c is the *cohesion intercept*, a characteristic of cohesive soils. Representative values of ϕ and c are given in Table 35.12.

Table 35.12 *Typical Strength Characteristics (above the water table)*

group symbol	cohesion (as compacted) c lbf/ft^2 (kPa)	cohesion (saturated) c_{sat} lbf/ft^2 (kPa)	effective stress friction angle ϕ
GW	0	0	> 38°
GP	0	0	> 37°
GM	–	–	> 34°
GC	–	–	> 31°
SW	0	0	38°
SP	0	0	37°
SM	1050 (50)	420 (20)	34°
SM-SC	1050 (50)	300 (14)	33°
SC	1550 (74)	230 (11)	31°
ML	1400 (67)	190 (9)	32°
ML-CL	1350 (65)	460 (22)	32°
CL	1800 (86)	270 (13)	28°
OL	–	–	–
MH	1500 (72)	420 (20)	25°
CH	2150 (100)	230 (11)	19°
OH	–	–	–

(Multiply lbf/ft^2 by 0.04788 to obtain kPa.)

18. TRIAXIAL STRESS TEST

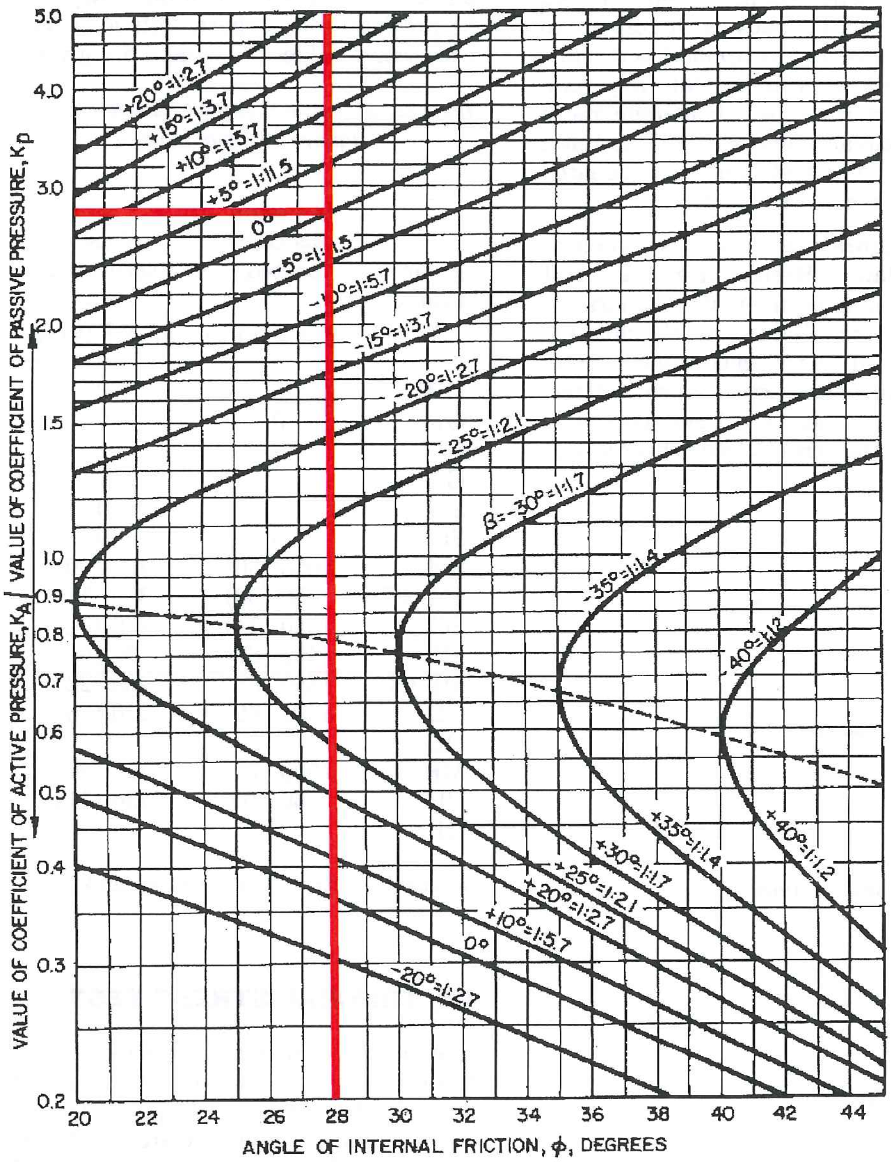
The *triaxial test* is a more sophisticated method than the direct shear test for determining the strength of soils. In the triaxial test apparatus, a cylindrical sample is stressed completely around its peripheral surface by pressurizing the sample chamber. This pressure is referred to as the *confining stress*. Then, the soil is loaded vertically to failure through a top piston. The confining stress is kept constant while the axial stress is varied. The radial component of the confining stress is called the *radial stress*, σ_R, and represents the minor principal stress, σ_3. The combined stresses at the ends of the sample (confining stress plus applied vertical stress) are called the *axial stress*, σ_A, and represent the major principal stress, σ_1.[14]

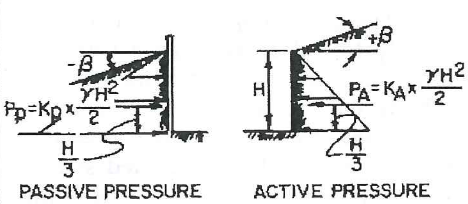
Results of a triaxial test at a given chamber pressure are plotted as a stress-strain curve. Two such examples are illustrated in Fig. 35.14. The axial component of

[13]In a physical sense, the angle of internal friction for cohesionless soils is the angle from the horizontal naturally formed by a pile. For example, a uniform fine sand makes a pile with a slope of approximately 30°. For most soils, the natural angle of repose will not be the same as the angle of internal friction, due to the effects of cohesion.
[14]In reality, the triaxial test apparatus is a "biaxial" device because it controls stresses in only two directions: radial and axial.

Geotechnical

M.V
CH





$$K_A=\left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\beta)}}\right]^2;\quad K_P=\left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi+\cos\phi\tan\beta)}}\right]^2$$

K_A & K_P = COEFFICIENTS FOR COULOMB'S EQUATION FOR ACTIVE AND PASSIVE EARTH PRESSURE (NO SHEAR STRESS ON VERTICAL PLANES).

P_A = ACTIVE RESULTANT
P_P = PASSIVE RESULTANT
γ = UNIT WEIGHT OF SOIL
H = HEIGHT OF WALL
ϕ = ANGLE OF INTERNAL FRICTION
β = SLOPE ANGLE

FIGURE 3
Active and Passive Coefficients, Sloping Backfill
(Granular Soils)

Appendix I. Report Copy List

Copy List: Lance Schrey, Quincy Engineering.

Appendix F - Hydraulic Report

Stevenson Bridge over Putah Creek Rehabilitation Project
Solano County, California
Federal-Aid Project No. BRLS-5923(059)
Existing Bridge No. 23C0092

Bridge Design Hydraulic Study Report

Prepared for:

Prepared by:

January 2018

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Stevenson Bridge over Putah Creek Rehabilitation Project
Solano County, California
Federal-Aid Project No. BRLS-5923(059)
Existing Bridge No. 23C0092

Bridge Design Hydraulic Study Report

Submitted to:
Solano County Department of Public Works

This report has been prepared by or under the supervision of the following Registered Engineer. The Registered Civil Engineer attests to the technical information contained herein and has judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

Han-Bin Liang, Ph.D., P.E.
Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
HAN-BIN LIANG
No. C 049404
Exp. 6/30/18
CIVIL
STATE OF CALIFORNIA

1/11/2018
Date

January 2018

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Executive Summary

The Solano County Department of Public Works (County) is proposing to rehabilitate the existing bridge on Stevenson Bridge Road over Putah Creek (Bridge No. 23C0092). The Stevenson Bridge over Putah Creek Rehabilitation Project (Project) is located approximately 5 miles west of the city of Davis and 8 miles east of the City of Winters.

The bridge geometrics were based on the survey information provided by Quincy Engineering in 2016 and the California Department of Transportation's (Caltrans) Bridge Inspection Report (BIR). The bridge has an opening of approximately 292 ft (abutment face to abutment face). The lowest soffit elevation is 91.2 ft. The Project proposes to rehabilitate and seismically retrofit the bridge to correct its deficiencies and realign the south approach of Stevenson Bridge Road.

The purpose of this Bridge Design Hydraulic Study Report is to summarize the hydrologic and hydraulic (H&H) modeling results of existing and proposed conditions with operation and maintenance (O&M), and 50-year and 100-year design flows. The "proposed bridge" refers to the rehabilitation of the existing bridge. The report also summarizes the potential design scour calculations and proposed rock slope protection (RSP) countermeasures for this Project.

The peak design flows for the Project were obtained from the Central Valley Flood Protection Board (CVFPB) and estimated using peak stream flow data from United States Geological Survey (USGS) gage station 1145400, which is located upstream of the Project site. The O&M flow of 40,000 cfs was provided by the CVFPB. The 100-year, and 50-year design flows were calculated to be 42,600 and 25,500 cfs, respectively.

The hydraulic analysis was performed using the U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) and a survey provided by Quincy Engineering, Inc. from 2016. The water surface elevation (WSEs) comparison among the O&M summarizes the 100-year, and 50-year peak design flows in the following table. The WSEs were found to be identical for the existing and proposed conditions.

Summary of Water Surface Elevations (Existing and Proposed)

River Station	Description	Water Surface Elevation (ft NAVD 88)		
		O&M	100-Year	50-Year
1224	640 ft upstream of existing bridge	85.1	86.1	78.7
922.8	330 ft upstream of existing bridge	84.7	85.7	78.4
686	97 ft upstream of existing bridge	84.3	85.3	78.0
600.3	11 ft upstream of existing bridge	84.3	85.3	78.0
575.7 BR U	Upstream face of existing bridge	83.6	84.5	77.5
575.7 BR D	Downstream face of existing bridge	83.6	84.6	77.5
548	15 ft downstream of existing bridge	83.6	84.6	77.5
290.7	270 ft downstream of existing bridge	83.2	84.2	77.1
0	560 ft downstream of existing bridge	82.9	83.8	76.7

A scour analysis was performed for the bridge using the 100-year design flow. Long-term, contraction, and local scour were evaluated using the methods outlined in the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular No. 18 (HEC-18), *Evaluating Scour at Bridges* (2012). The following table summarizes the estimated scour depths for the bridge at the Project site.

Total Scour Table

Location	Contraction Scour (ft)	Local Scour (ft)	Long-Term Scour (ft)	Total Scour (ft)
Abutment 1	0.0	--	5.1	5.1
Pier 2	0.0	16.1	5.1	21.2
Pier 3	0.0	20.6	5.1	25.7
Pier 4	0.0	12.1	5.1	17.2
Abutment 5	0.0	--	5.1	5.1

RSP is proposed at the abutment of the bridge to protect the banks and reduce erosion potential. The median diameter of the RSP for the bridge abutments was calculated using the Isbash relationship from HEC-23, *Bridge Scour and Stream Instability, Design Guideline 14* (FHWA). Class IV RSP is proposed along with Class 8 RSP fabric.

Flow data from USGS gaging station 11454000 was extracted for the construction period from June 1 to October 15 for the construction season flow analysis. The minimum, average, and maximum peak flows were calculated based on the extracted data from 1988 through 2017. By assuming little to no precipitation during the construction period, the statistical analysis results for the construction season flow will be the same for the gaging station and Project site. The contractor may elect to work later in the season when flows are lower with the appropriate diversion system to move flows away from the necessary work area. See Section 6 and Appendix E for additional flow information for use in the design of the diversion system.

Acronyms

BIR	Bridge Inspection Report
Caltrans	California Department of Transportation
cfs	cubic feet per second
County	County of Solano Department of Public Works
CVFPB	Central Valley Flood Protection Board
D50	median stone diameter
ESRI	Environmental Systems Research Institute
FHWA	Federal Highway Administration
ft	feet
HDM	Highway Design Manual
HEC-18	Hydraulic Engineering Circular No. 18
HEC-23	Hydraulic Engineering Circular No. 23
HEC-RAS	Hydrologic Engineering Centers River Analysis System
mm	millimeter
NAVD 88	North American Vertical Datum of 1988
O&M	operation and maintenance
Project	Stevenson Bridge over Putah Creek Bridge Rehabilitation Project
RS	river station
RSP	rock slope protection
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WSE	water surface elevation

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1 GENERAL DESCRIPTION

The Solano County Department of Public Works (County), in conjunction with the County of Yolo, the California Department of Transportation (Caltrans), and the Federal Highway Administration (FHWA), is proposing to rehabilitate Bridge 23C0092 at Stevenson Bridge Road (Stevenson Bridge) over Putah Creek. The Stevenson Bridge over Putah Creek Bridge Rehabilitation Project (Project) is located approximately 5 miles west of the City of Davis and 8 miles east of the City of Winters. See Figure 1 for the Project location map, Figure 2 for the Project vicinity map, and Figure 3 for the Project aerial map.

1.1 Project Description

The Project proposes to rehabilitate and seismically retrofit Stevenson Bridge Road bridge (Stevenson Bridge) to correct its deficiencies and realign the south approach of Stevenson Bridge Road. Additional proposed Project activities include a staging area, construction of an access road, a temporary creek crossing, stream diversion, a traffic detour, and utility relocation.

The purpose of this Project is to improve public safety by rehabilitating the seismically vulnerable and scour critical structure. Also proposed are additional safety features, which include roadway alignment improvements, and repair of the existing concrete railing.

1.2 Existing Bridge

The existing Stevenson Bridge (Bridge No. 23C0092) at Stevenson Bridge Road was constructed in 1923. The existing roadway is functionally classified as a major collector, which provides access for approximately 900 vehicles per day between Solano and Yolo counties. The structure is comprised of reinforced concrete T-beam approach spans and concrete tied arch main spans. The bridge structure is approximately 296 feet (ft) long and 24 ft wide with two 40-foot approach spans and two 108-ft tied arch main spans. The substructure is supported on reinforced concrete piers with curtain walls, founded on timber or concrete pile foundations. The abutments are founded on spread footings.

The Stevenson Bridge, or the “Graffiti Bridge” as it is known locally, has considerable public and historical interest. The bridge is one of three tied arch bridges in northern California and is considered historically significant. The same plans were used to construct the Rumsey Bridge located approximately 40 miles to the northwest. The Rumsey Bridge is currently scheduled to be replaced, which only increases the historical importance of the Stevenson Bridge.

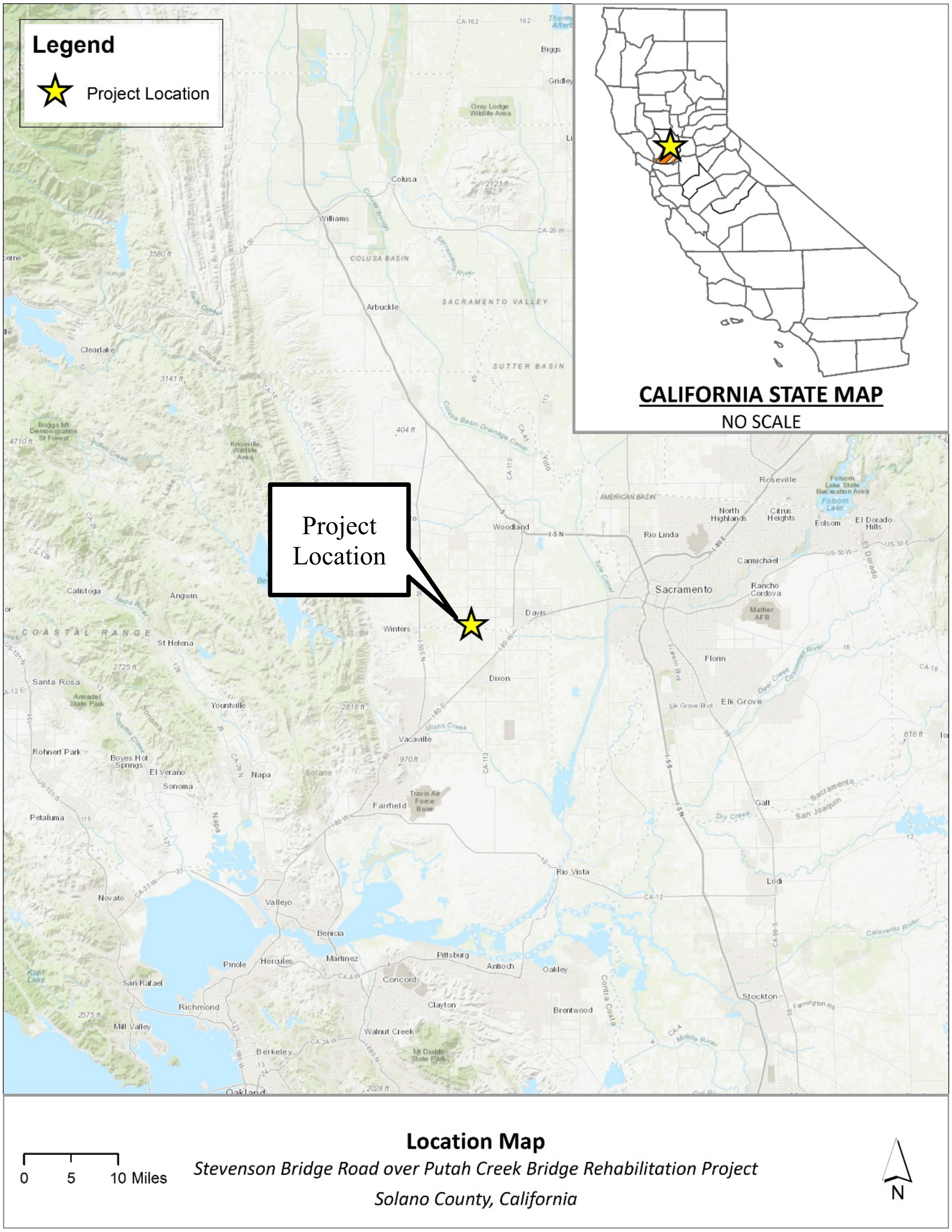

Figure 1. Project Location Map

Source: Environmental Systems Research Institute (ESRI)

Figure 2. Project Vicinity Map

Source: ESRI

Figure 3. Project Aerial Map

Source: ESRI

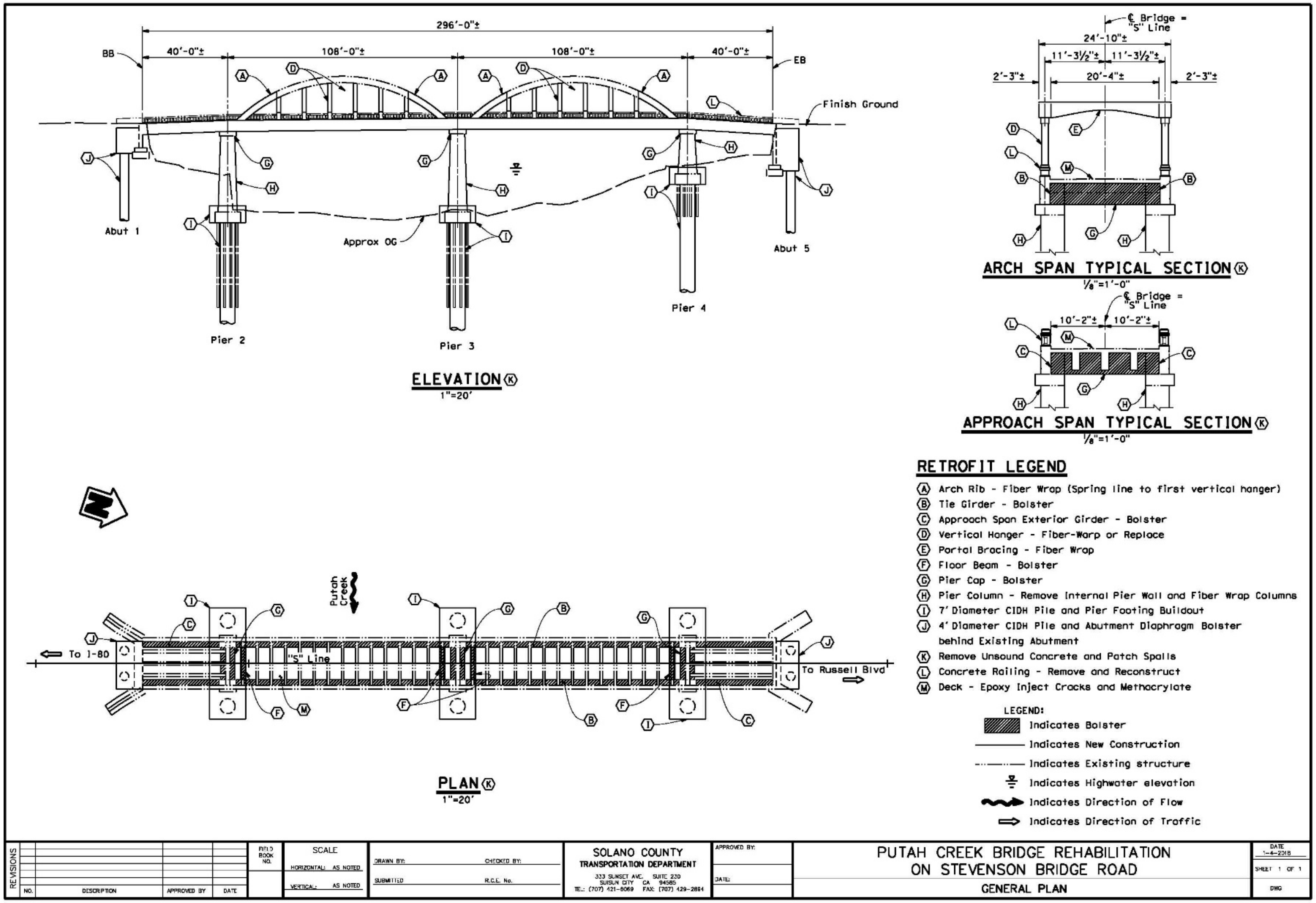

Figure 4. General Plan

Source: Quincy Engineering, Inc.

1.3 Proposed Bridge Rehabilitation

The rehabilitation of the existing bridge, which affects the hydraulics, will consist of large diameter cast-in-drilled-hole (CIDH) piles. It may require large shored excavation to strengthen the existing foundation and tie the large CIDH foundation to the existing footing. The new footing will be approximately 17 ft in width, 52 ft in length, and 8 ft in thickness. See Figure 4 for the general plan.

1.4 Purpose

The purpose of this Bridge Design Hydraulic Study report is to present the design flow characteristics for the existing and proposed bridges. The "proposed bridge" refers to the rehabilitation of the existing bridge. This report provides the calculated scour potential and recommendations on the need for scour countermeasures for the proposed bridge. This report presents the hydraulic characteristics and scour potential and recommendations on the need for scour countermeasures for the proposed bridge.

1.5 Key Tasks

Key tasks performed in this study included: 1) a review of available hydrologic data, 2) a hydrologic study, 3) a hydraulic analysis to determine design water surface elevations (WSEs) and flow velocities for the existing and proposed bridges, 4) a scour analysis to estimate potential scour depths for the proposed bridge, and 5) scour countermeasure analyses and recommendations for the proposed bridge.

1.6 Design Criteria

The following criteria are applicable for the Project and were considered for the design of the rehabilitation elements for the proposed bridge.

1.6.1 Hydraulic Design Criteria

1.6.1.1 FHWA Standards

The FHWA criterion refers to the California amendments to American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (2014), which indicates that the proposed bridge profile should provide adequate freeboard to pass anticipated drift for the 50-year design flood, to pass the 100-year base flood without freeboard, or the flood of record without freeboard, whichever is greater.

1.6.1.2 Caltrans Standards

The Caltrans criteria for the hydraulic design of bridges is that they be designed to pass the 2% probability of annual exceedance flow (50-year design discharge) or the flood of record, whichever is greater, with adequate freeboard to pass anticipated drift. Two feet of freeboard is commonly used in bridge designs. The bridge should also be designed to pass the 1% probability of annual exceedance flow (100-year design discharge, or base flood). No freeboard is added to the base flood.

1.6.1.3 Central Valley Flood Protection Board Standards

Because the Project is located within the Central Valley Flood Protection Board's (CVFPB) jurisdiction, the bridge freeboard criteria for CVFPB are determined by the design capacity and number of residents in the Project vicinity. The soffit of the proposed bridge must be at least 3 ft above the design flood profile for major streams (channel capacity greater than 8,000 cfs ft per second [cfs]). The required freeboard can be reduced to 2 ft for minor streams (design capacity less than 8,000 cfs) where significant amounts of stream debris are unlikely. The CVFPB will require a 200-year level of protection starting in 2025 for urban and urbanizing areas in the California Central Valley. A design flood can be the 100-year flow in non-urban areas.

1.6.1.4 Solano County Standards

Per Solano County's *Road Improvement Standards and Land Development Requirements* (2006), bridges shall be designed to pass a 50-year storm with a minimum of 2 ft of freeboard, and pass a 100-year storm with no freeboard. Streams that carry large floating debris may require greater freeboard.

1.6.2 Scour Design Criteria

The evaluation of potential scour at the proposed bridge followed the criteria described in the FHWA's Hydraulic Engineering Circular No. 18 (HEC-18), "*Evaluating Scour at Bridges*" (Fifth Edition). The evaluation of potential scour was based on the hydraulic characteristics of the 100-year design discharge. The total scour was estimated based upon the cumulative effects of the long-term bed elevation change, general (contraction) scour, and local scour. The life expectancy of the bridge was considered in determining the long-term bed elevation change of the waterway; it was based on an assumed 50-year design life for a retrofit bridge.

1.7 Vertical Datum

The Project references the North American Vertical Datum of 1988 (NAVD 88).

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2 GEOGRAPHIC SETTING

2.1 Geographic Location

The Project is located in Solano County near the border of Yolo County at 38°32’11.3” North latitude and 121°51’3.9” West longitude and is approximately 5 mi west of the City of Davis.

2.2 Watershed Description

According to StreamStats, Putah Creek drains a watershed area of approximately 644 square miles at Stevenson Bridge (see Figure 5). The headwaters are located in the Vaca Mountains, and the Monticello Dam in Vaca Mountains forms Lake Berryessa approximately 16 mi upstream from the Project site. After crossing the Project site, Putah Creek flows approximately 5 mi east toward the City of Davis.

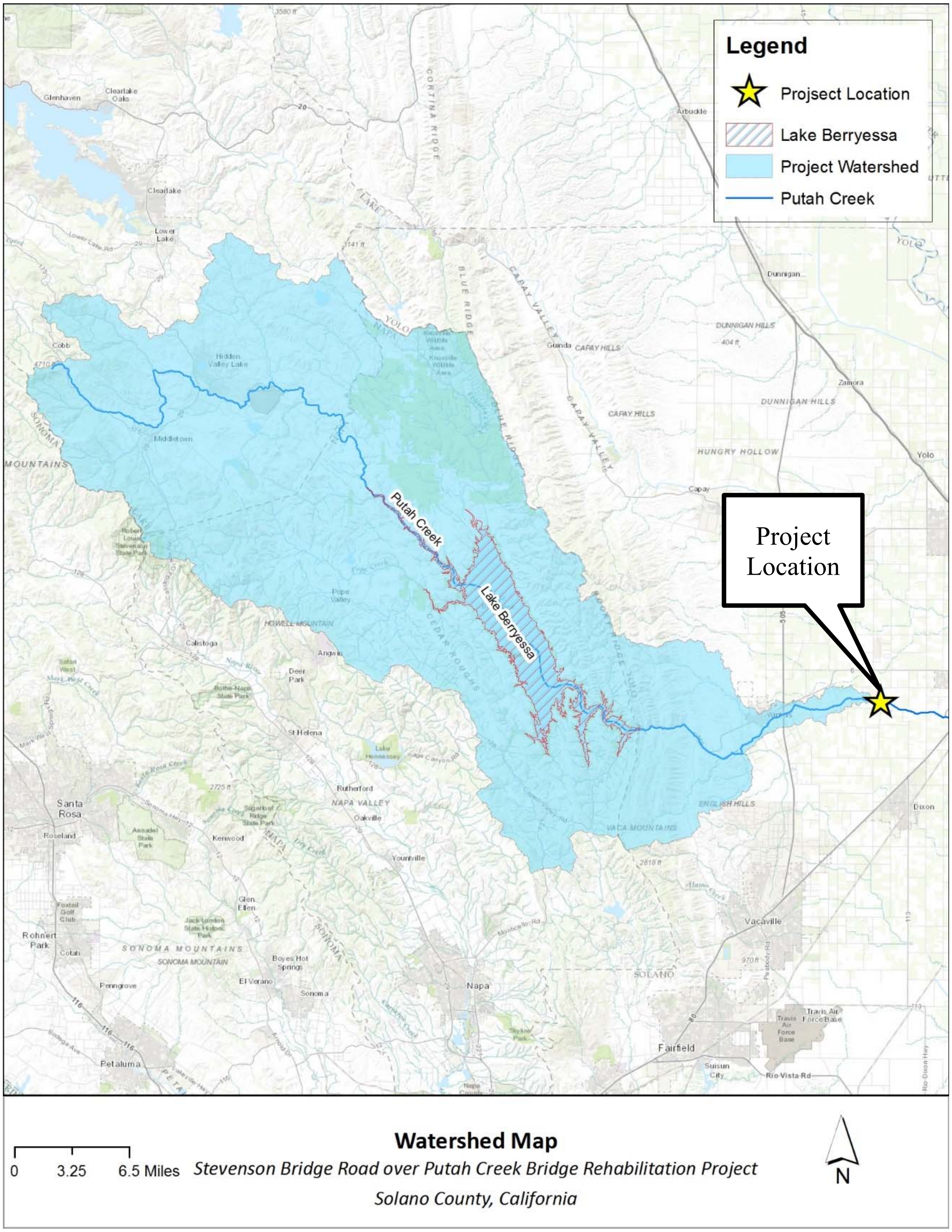

Figure 5. Project Watershed Map

Source: ESRI

3 HYDROLOGIC ANALYSIS

The following sub-sections describe the hydrologic data sources that were used to estimate the flows for the Project site.

3.1 Hydrologic Design Methods

WRECO evaluated the hydrology at the Project site using the following hydrologic design methods:

1. Peak Streamflow Statistical Analysis of Gaging Station Data
2. Central Valley Flood Protection Board Operation and Maintenance (O&M) Flow

Both hydrologic design methods are described in the following sections and are adopted for the Project.

3.2 Design Discharge Summary

3.2.1 Peak Streamflow Statistical Analysis of Gaging Station Data

The design flows for Putah Creek were estimated using peak stream flow data from United States Geological Survey (USGS) gaging station 11454000, which is located just downstream of Monticello Dam. Figure 6 shows the location of the gaging stations nearest to the Project site. The USGS gaging station 11454000 includes 86 annual peak flow measurements taken from water years 1931 through 2016 (see Figure 7, which shows a graph of the peak annual flow data points). Per the USGS National Water Information System, the drainage area at the gaging station is 574 square miles (sq. mi).

According to California Data Exchange Center, Monticello Dam was constructed in 1957, and the statistical analysis used the peak stream flow data from water years 1957 through 2016 to cover the regulated flow period.

A flood frequency analysis was performed to predict the peak design flows using the observed annual peak flow data from USGS gaging station 11454000. The observed annual peak flow discharge data were used to calculate the statistical variables by using PEAKFQ and following the Bulletin 17B methodologies (U.S. Interagency Advisory Committee on Water Data 1982). The Bulletin 17B method of analysis utilizes the Log-Pearson Type III distribution as a base method for the frequency analysis and also incorporates the use of several additional parameters, including a regional skewness and skewness of the station record sample data. By doing so, the Bulletin 17B procedures are more robust than simply fitting the Log-Pearson Type III distribution to the peak flow record. The design flows from the PEAKFQ analysis are then adjusted based on the ratio of the drainage area between the Project site and the gaging station. The estimated 50- and 100-year design flows at the Project site are 25,500 and 42,600 cfs, respectively.

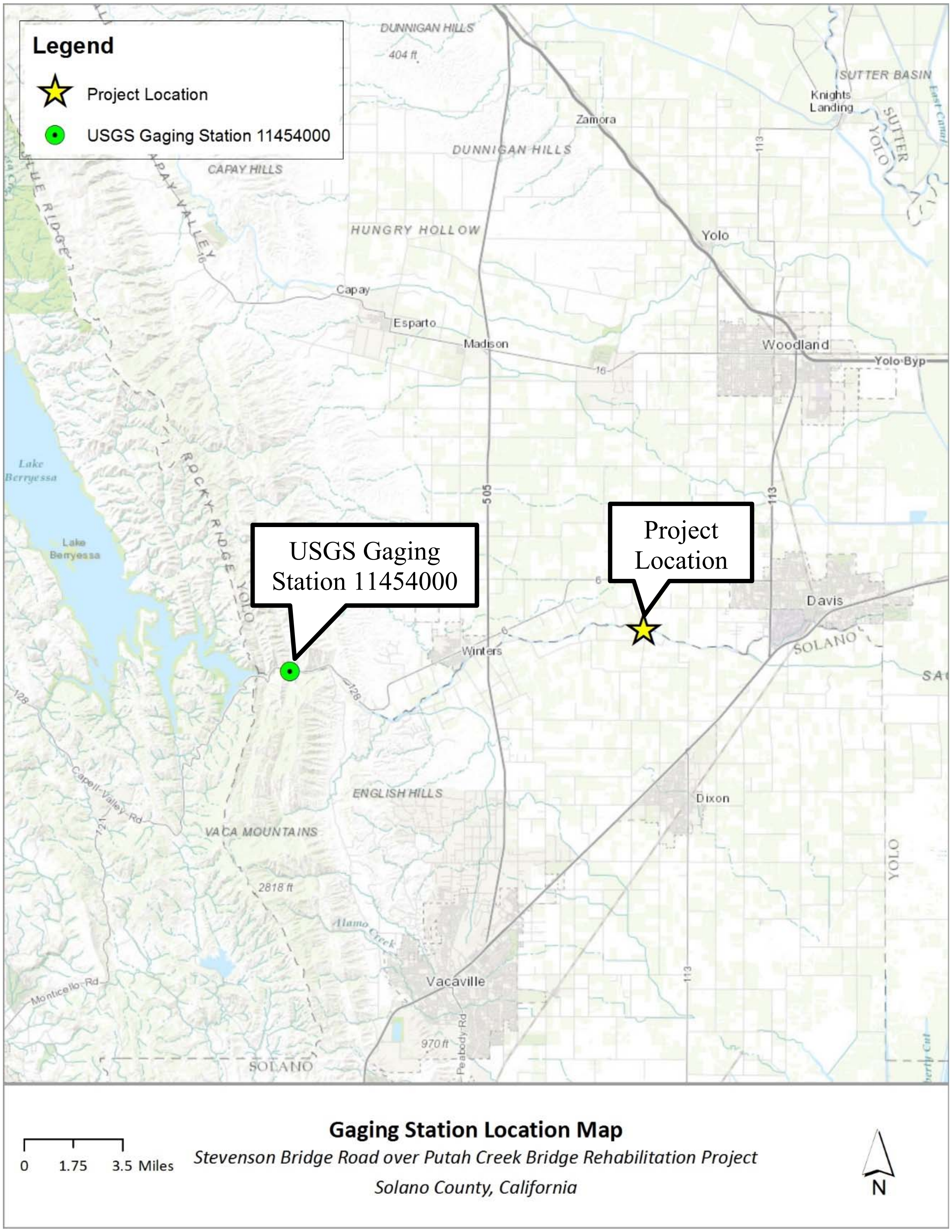

Figure 6. Gaging Station Location Map

Source: ESRI

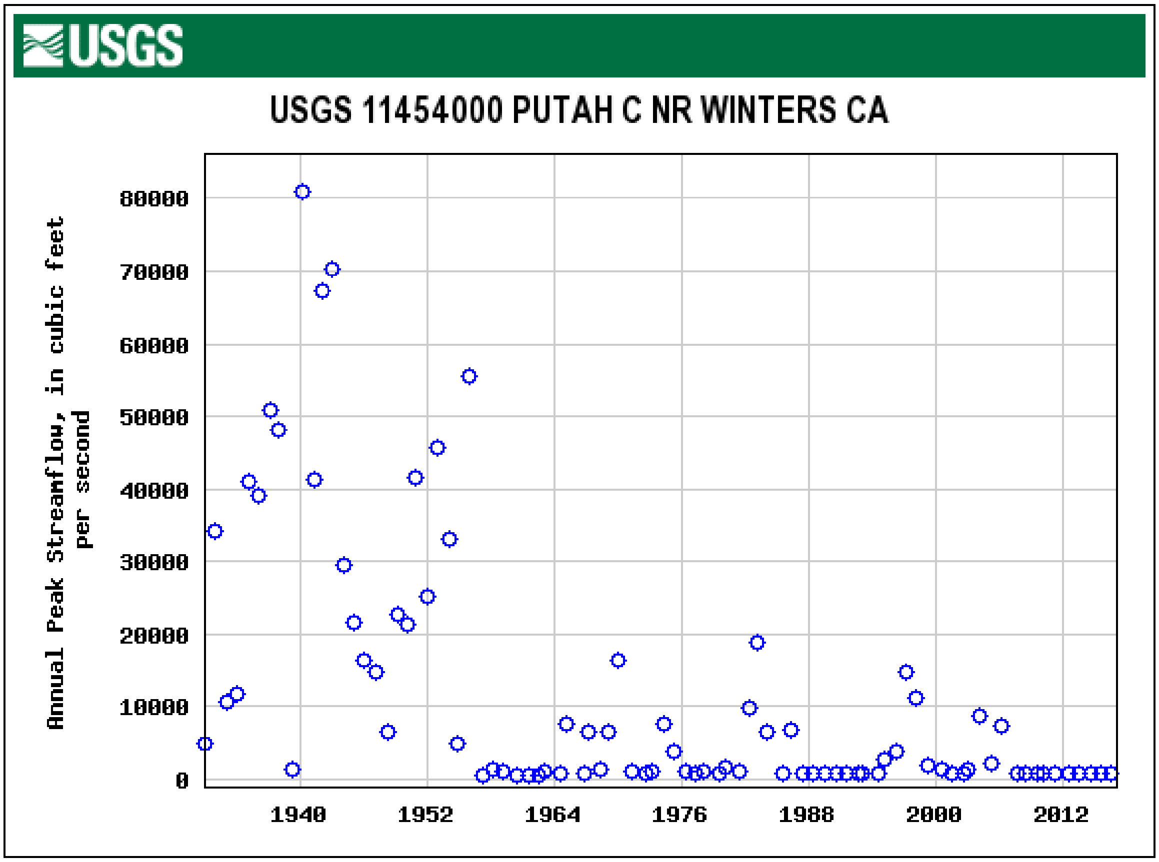

Figure 7. Putah Creek near Winters CA (USGS Gaging Station 11454000) Peak Annual Flow Record

Source: USGS

3.2.2 Central Valley Flood Protection Board Operation and Maintenance Flow

Per discussion with the CVFPB on November 2, 2016, the O&M design flow at Putah Creek is 40,000 cfs.

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4 HYDRAULIC ANALYSIS

The following sections discuss the development of the hydraulic models and summarize the results for the existing and proposed conditions. The water surface profile plots, hydraulic summary tables, and channel cross sections are included in 0 for the existing bridge and Appendix B for the proposed bridge.

4.1 Design Tools

The hydraulic analyses were performed for the existing and proposed conditions using the United States Army Corps of Engineers' (USACE) Hydrologic Engineering Centers River Analysis System (HEC-RAS) modeling software, Version 5.0.3.

4.2 Cross Section Data

The cross-sectional channel geometry for the hydraulic model was developed using survey data provided by Quincy Engineering, Inc. from 2016. The survey references the NAVD 88 datum with an unknown horizontal datum. The seven surveyed cross sections extend approximately 650 ft upstream and 600 ft downstream of Stevenson Bridge measured along Putah Creek (see Figure 8, which shows the locations of the cross sections). The cross section naming convention is by river stations (RS) with the cross section number increasing in RS going upstream.

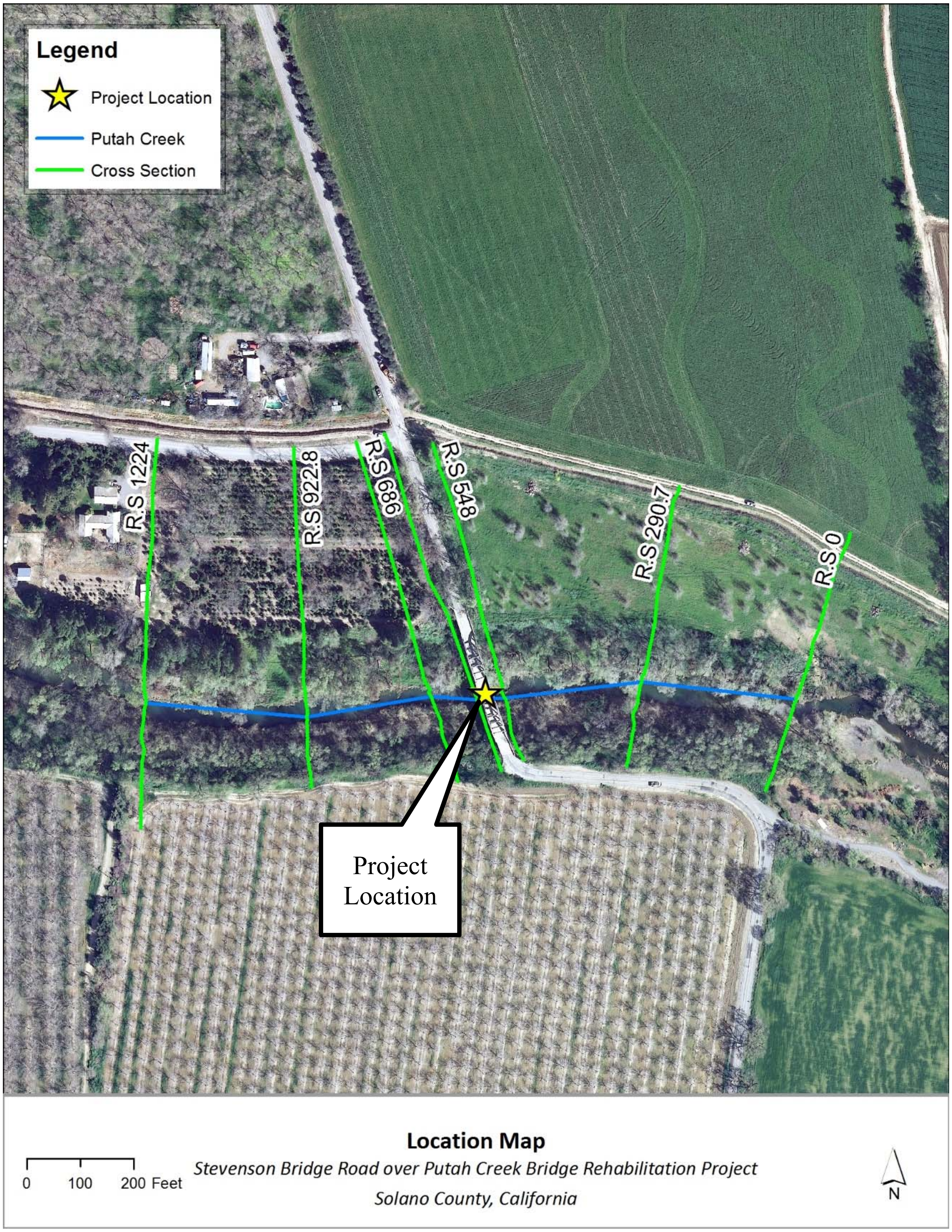


Figure 8. Cross Section Locations

Source: ESRI

4.3 Modeled Hydraulic Structures

The geometry of the existing bridge in the hydraulic model was based on the survey data provided by Quincy Engineering, Inc. in 2016 and Caltrans' Bridge Inspection Report (BIR). The existing bridge has an opening of approximately 292 ft (abutment face to abutment face). The lowest soffit elevations is 91.2 ft. The design elements of the rehabilitation (as shown in Figure 4) that are within the limits of the design flood elevations were modeled.

4.4 Model Boundary Condition

A normal depth of 0.0014 ft/ft was used as the downstream reach boundary condition, and it was based on thalweg elevations from the USGS topographic survey of Putah Creek downstream of the bridge.

4.5 Manning's Roughness Coefficients

Manning's roughness coefficients were used in the hydraulic model to estimate energy losses in the flow due to friction. A roughness coefficient of 0.035 was used to describe the low flow channel, and a roughness coefficient of 0.065 was used to describe the overbank areas. These values were selected based on aerial imagery in the Project vicinity. The channel in the vicinity of Stevenson Bridge Road is shown in Photo 1, which was taken on June 6, 2016 when the Project Team visited the Project site.

Photo 1. Putah Creek in Vicinity of Stevenson Bridge Road

4.6 Expansion and Contraction Coefficients

Expansion and contraction coefficients were used in the hydraulic model to represent energy losses in the channel. An expansion coefficient of 0.3 and a contraction coefficient of 0.1 were used to represent the channel in the vicinity of Stevenson Bridge. These values represent a channel with a gradual transition between cross sections.

4.7 Water Surface Elevations

The WSEs at the locations just upstream and downstream of the bridge for the existing condition are summarized in Table 1. The cross section sat the upstream sides of the bridges are shown in Figure 9 for the existing bridge. The water surface profiles along the studied stream reach are presented in Figure 10 for the existing condition. The HEC-RAS calculations for the existing bridge can be found in 0. The design elements of the rehabilitation were modeled and did not affect the WSEs.

Table 1. Putah Creek Water Surface Elevations Comparison (Existing & Proposed)

River Station	Description	Water Surface Elevation (ft NAVD 88)		
		O&M	100-Year	50-Year
1224	640 ft upstream of existing bridge	85.1	86.1	78.7
922.8	330 ft upstream of existing bridge	84.7	85.7	78.4
686	97 ft upstream of existing bridge	84.3	85.3	78.0
600.3	11 ft upstream of existing bridge	84.3	85.3	78.0
575.7 BR U	Upstream face of existing bridge	83.6	84.5	77.5
575.7 BR D	Downstream face of existing bridge	83.6	84.6	77.5
548	15 ft downstream of existing bridge	83.6	84.6	77.5
290.7	270 ft downstream of existing bridge	83.2	84.2	77.1
0	560 ft downstream of existing bridge	82.9	83.8	76.7

Notes: ft = feet
NAVD 88 = North American Vertical Datum of 1988
BR U = bridge upstream face
BR D = bridge downstream face

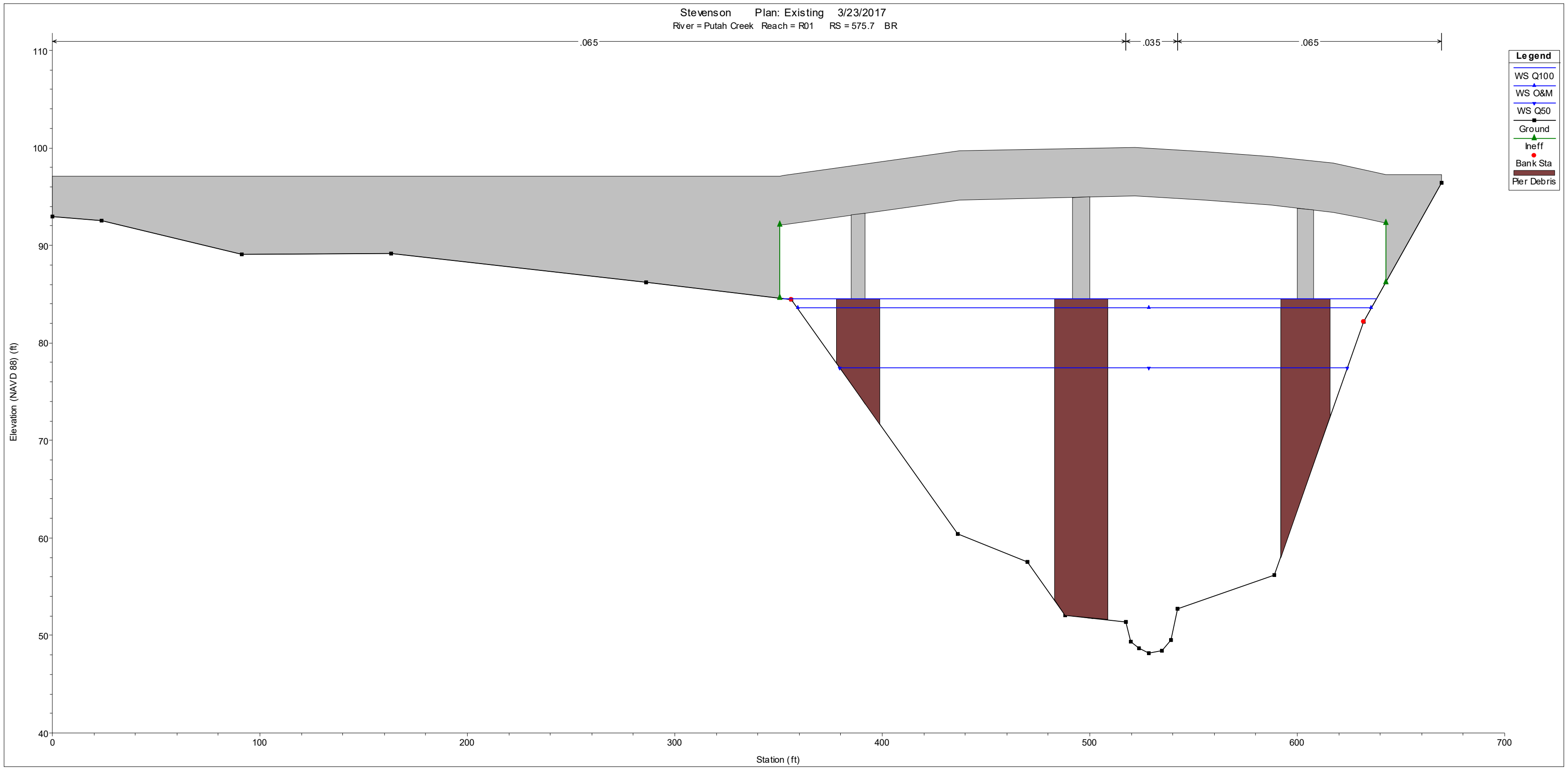


Figure 9. Upstream Face of Bridge (Existing and Proposed), Looking Downstream (South)

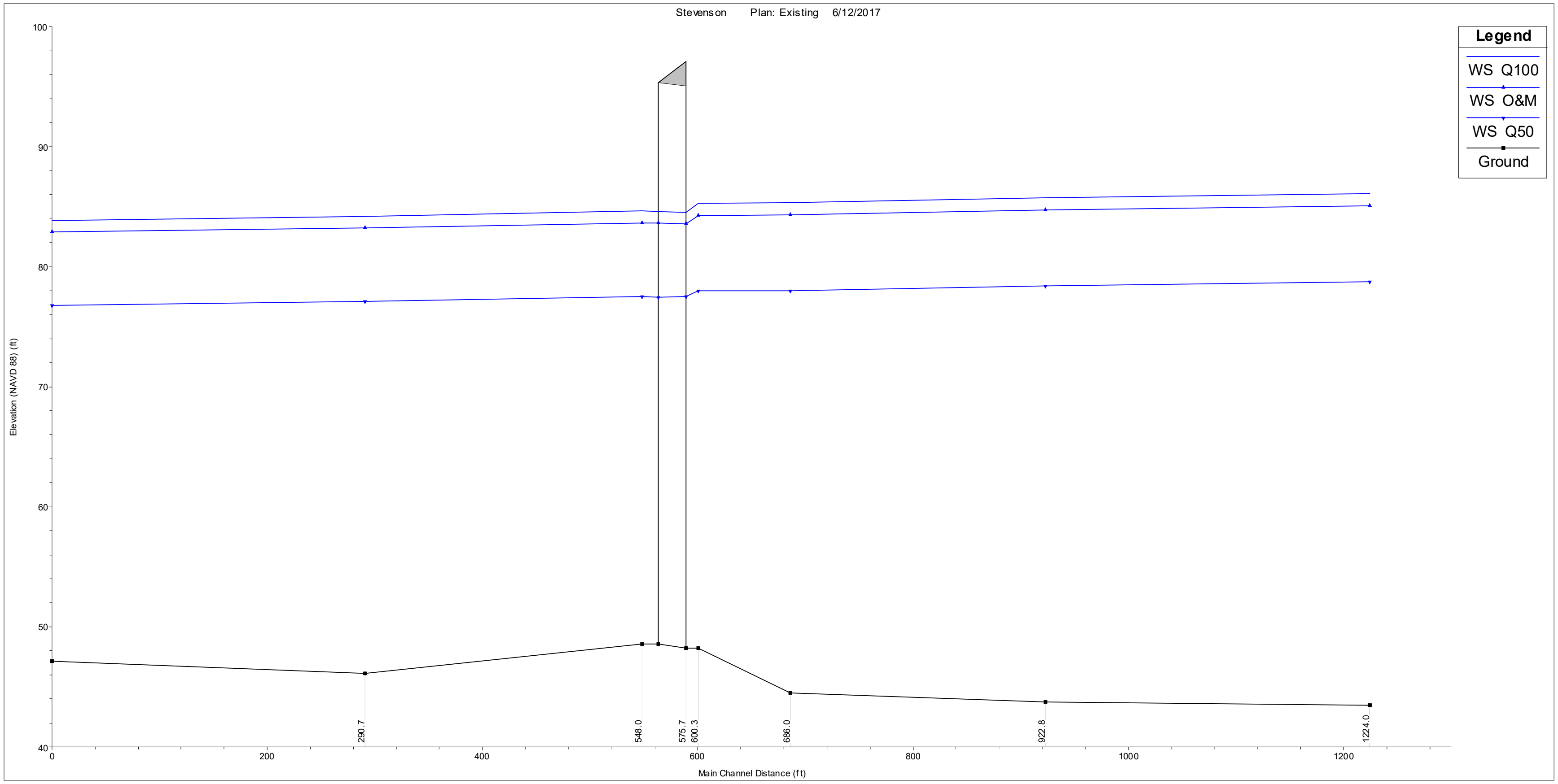


Figure 10. Putah Creek Water Surface Profiles at Stevenson Bridge (Existing and Proposed)

4.8 Freeboard

The freeboard requirements applicable to the Project are discussed in Section 1.6.1. To summarize, the FHWA requires that the bridge be designed to pass the 50-year storm event with adequate freeboard to account for debris and bedload. Caltrans also requires that the bridge be designed to pass the 50-year storm event with adequate freeboard to account for debris and bedload (Caltrans recommends 2 ft of freeboard), or the 100-year storm event with no freeboard. The CVFPB requires the bridge be design to pass the O&M flow with 3 ft of freeboard. Solano County has the same design criteria as Caltrans. The minimum soffit elevations and available freeboard for the bridges are presented in Table 2 for existing bridge. The existing bridge meets the applicable design criteria. With the rehabilitation improvements, the bridge will maintain the same freeboard.

Table 2. O&M Flow Water Surface Elevations and Freeboard

Design Flow	Soffit Elevation (ft NAVD 88)	WSE (ft NAVD 88)	Freeboard (ft)
O&M	91.2	84.3	6.9
100-Year		85.3	5.9
50-Year		78.0	14.1

Notes: ft = feet
NAVD 88 = North American Vertical Datum of 1988

4.9 Flow Velocities

The average channel flow velocities were estimated for the existing and proposed conditions from the developed hydraulic models, which are summarized in Table 3. The proposed rehabilitation improvements did not impact the average channel velocities.

Table 3. Summary of the Average Channel Velocities Comparison

River Station	Description	Average Channel Velocities (ft/s)		
		O&M	100-Year	50-Year
1224	640 ft upstream of existing bridge	6.3	6.5	5.4
922.8	330 ft upstream of existing bridge	6.4	6.5	5.5
686	97 ft upstream of existing bridge	7.0	7.1	6.0
600.3	11 ft upstream of existing bridge	6.5	6.6	5.6
575.7 BR U	Upstream face of existing bridge	8.6	8.8	7.3
575.7 BR D	Downstream face of existing bridge	7.6	7.8	6.6
548	15 ft downstream of existing bridge	7.1	7.2	6.1
290.7	270 ft downstream of existing bridge	7.3	7.5	6.3
0	560 ft downstream of existing bridge	6.8	6.9	5.9

Notes: Br. U = Bridge Upstream
Br. D = Bridge Downstream

5 SCOUR ANALYSIS

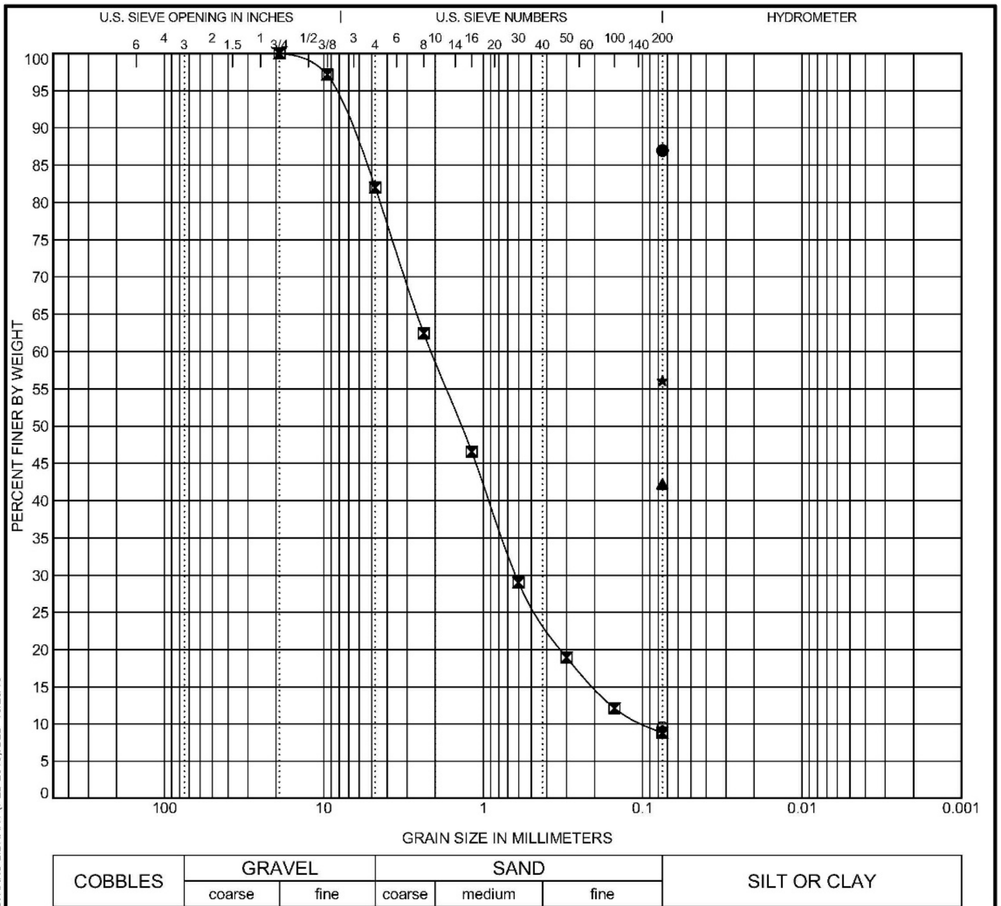
WRECO evaluated bridge scour per the criteria described in HEC-18, *Evaluating Scour at Bridges* (FHWA 2012). The minimum design criterion for bridge scour is the 100-year design storm. WRECO evaluated the scour potential and scour countermeasure analysis using the results of the steady-state flow analysis from HEC-RAS. The scour calculations assume that the channel bed material is erodible. The following sub-sections summarize the results of the analysis.

5.1 Caltrans Bridge Inspection Reports

Available BIRs were reviewed for relevant scour information. The March 28, 2013 bridge inspection was performed when the water was flowing only under Span 2, which allowed all visible elements to be fully inspected. The BIR noted that the pile cap at Bent 3 had a 58-inch vertical exposure but no undermining during the inspection. Moreover, the bridge was assigned a National Bridge Inventory Item 113, scour critical bridge rating of “3,” which represents that the bridge is scour critical, and bridge foundations were determined to be unstable for the assessed or calculated scour condition. The 2011, 2009, and 2008 BIRs also note a similar scour condition to that recorded in the 2013 BIR.

5.2 Existing Channel Bed

The contraction and local scour calculations were based on the flow characteristics from the hydraulic model for the 100-year peak flow and the grain size distribution from the particle size analysis. Based on the particle size analysis performed by Cal Engineering and Geology in 2016, the median grain size diameter (D_{50}) was approximately 1.2 millimeters (mm), and the channel bed material exhibits cohesive properties (see Figure 11).



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● B-1	79.5						
⊠ B-2	6.5					1.89	21.86
▲ B-2	14.5						
★ B-2	19.5						
⊙ B-2	39.5						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1	79.5	0.075						87.0	
⊠ B-2	6.5	10	2.12	0.623	0.097	18.0	73.2	8.8	
▲ B-2								42.2	
★ B-2								56.1	
⊙ B-2								9.5	

CE&G
CAL ENGINEERING & GEOLOGY

Department of Transportation
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design - Geotech Support

GRAIN SIZE DISTRIBUTION

DIST.	COUNTY	ROUTE	POSTMILE	EA
04	Solano			04-160600

PROJECT OR BRIDGE NAME: Quincy - Stevenson Rd. Bridge

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
	K. D		2 of 2

Figure 11. Grain Size Distribution

Source: Cal Engineering and Geology, Inc.

5.3 Long-Term Bed Elevation Change

Channel bed elevation may fluctuate over time as a result of changes in local sediment transport capacity and availability. Aggradation at the bridge site is a result of the deposition of material eroded from the channel when more sediment is supplied by watershed erosion and upstream channel flow than can be transported locally.

Degradation at the Project site is a result of scour of the channel due to sediment deficit. Only channel degradation is accounted for in the scour calculation.

The long-term bed elevation changes (long-term bed degradation) are typically based on historical channel data at the bridge site. Historical stream measurements that were recorded in the Caltrans BIRs were taken at the bridge and were included in the 1993, 2007, and 2015 BIRs (see Figure 12). Based on the stream measurements included in the BIRs and the 2016 survey information provided by Quincy Engineering, Inc., the long-term bed degradation projection is approximately 5.1 ft with a 50-year design life.

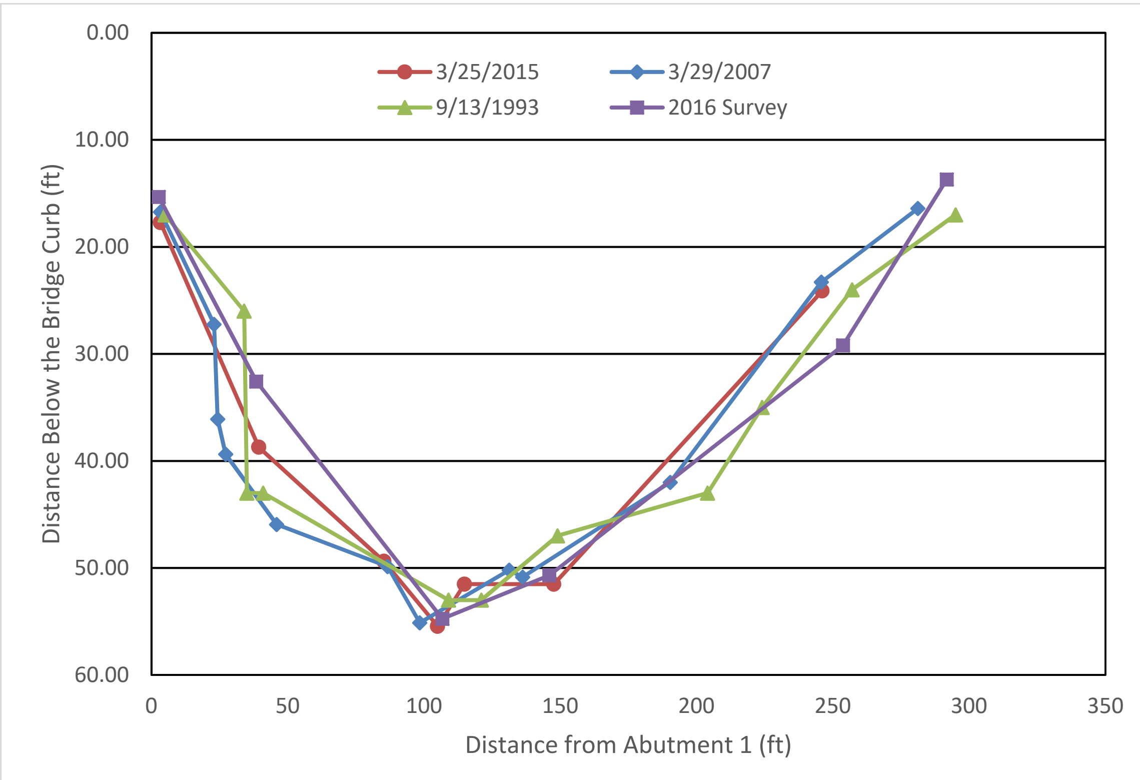


Figure 12. Historical Stream Measurements

5.4 Contraction Scour

Contraction scour occurs when the flow area of a stream is reduced by 1) the natural contraction of the stream channel, 2) by a bridge structure, or 3) the overbank flow forced back to the channel by roadway embankments at the roadway approach to a bridge. From the continuity equation, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction section, and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases. Thus, the velocity and shear stress decrease until relative equilibrium is reached, (i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach) or the bed shear stress is decreased to a value such that no sediment is transported out of the reach. Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width (FHWA 2012).

Based on the hydraulic model, the top widths of the cross sections increase from upstream (RS 686.0) to just upstream (RS 600.3) of the bridge. Because the channel does not contract, there is no contraction scour at the Project site.

5.5 Pier Scour

Pier scour is caused by vortices forming at the base of the pier. The scour depth at the pier is influenced by pier design, flow characteristics (flow rate and local velocity at the pier), and sediment particle size distribution.

For piers in cohesive materials, pier scour is more dependent on soil properties, and the HEC-18 recommends an equation presented by Briaud et. al. (2011):

$$y_s = 2.2K_1K_2a^{0.65}\left(\frac{2.6V_1 - V_c}{\sqrt{g}}\right)^{0.7}$$

Where:

y_s = scour depth, ft
K_1 = correction factor for pier nose shape; 1.1 for square nose, 1.0 for round nose, circular cylinder and group of cylinders, and 0.9 for sharp nose
K_2 = correction factor for angle of attack; 1.0 when angle is 0 degrees
a = pier width, ft
V_1 = mean velocity of flow directly upstream of the pier, ft/s
V_c = critical velocity for initiation of erosion of the cohesive material, ft/s
g = acceleration due to gravity, ft/s^2

For all piers, the velocity of the flow directly upstream of the pier was obtained from the HEC-RAS model using a velocity distribution. The local pier scour depths for the Project bridge over Putah Creek are summarized in Table 4.

Table 4. Local Pier Scour Depths

Location	Local Pier Scour (ft)
Pier 2	16.1
Pier 3	20.6
Pier 4	12.1

5.6 Abutment Scour

Abutment scour occurs when the bridge abutments block approaching flow. Because the abutments of the bridge will not block approaching flow with the design flow, there will be no local abutment scour associated with the bridge. However, armoring the banks at the abutments is necessary to prevent bank erosion, which would expose the abutments to scour.

5.7 Total Scour

Total scour is the sum of the local scour, contraction scour, and long-term bed elevation change. The total scour depths are summarized in Table 5. The scour depths were based on the cohesive soil equations. The detailed calculations are included in Appendix C.

Table 5. Scour Depths – Cohesive Soils

Location	Contraction Scour (ft)	Local Scour (ft)	Long-Term Scour (ft)	Total Scour (ft)
Abutment 1	0.0	--	5.1	5.1
Pier 2	0.0	16.1	5.1	21.2
Pier 3	0.0	20.6	5.1	25.7
Pier 4	0.0	12.1	5.1	17.2
Abutment 5	0.0	--	5.1	5.1

According to the *Bridge Memo to Designers,* bridge footings supported on soil or degradable rock should be embedded below the maximum computed scour depth or protected with a scour countermeasure, and the bridge foundations should not fail due to scour from the 100-year flow (Caltrans 2003). The bridge foundations should be designed to support the bridge with no lateral support down to the thalweg elevation minus the total scour depth, unless the risk of thalweg migration and local scour can be mitigated with a properly designed scour countermeasure.

According to the Caltrans memorandum dated October 23, 2015, *Scour Data Table on Foundation Plan*, a scour data table should also present a long-term scour elevation based upon the sum of the local scour depth. For the abutments, because rock slope protection (RSP) will be provided at the abutment embankment slopes, the scour elevations were based on the finished grade elevations, which are 79.5 and 81.1 ft for Abutment 1 and Abutment 5, respectively. The scour data is presented in Table 6.

Table 6. Scour Data Table for the Proposed Bridge

Location	Ground Elevation* (ft)	Long-Term (Degradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Abutment 1	84.6	79.5	--
Pier 2	48.2	43.1	16.1
Pier 3	48.2	43.1	20.6
Pier 4	48.2	43.1	12.1
Abutment 5	86.2	81.1	--

* The thalweg elevation is currently 48.2 ft NAVD 88

5.8 Scour Countermeasures

Two procedures for determining the RSP design for the proposed bridge were considered: Hydraulic Engineering Circular No. 23 (HEC-23), *Bridge Scour and Stream Instability* (FHWA 2009) and *Highway Design Manual* (HDM) (Caltrans 2016). RSP generally consists of rocks placed on channel and structure boundaries to limit the effects of erosion. It is the most common type of scour countermeasure due to its general availability, ease of installation, and relatively low cost.

The RSP design was calculated following procedures outline in HEC-23 and the HDM. The HEC-23 method results in a larger rock size (compared to the HDM) and is presented in the following discussion. The median stone diameter (D_{50}) of the RSP at the bridge abutment was calculated using the Isbash relationship.

The following equation was used to determine the D_{50} required for the proposed riprap erosion-control system to protect the channel-bank slope under the bridge:

For Froude number ($V/(gy)^{0.5}$) ≤ 0.80 (HEC-23, Isbash relationship):

$$\frac{D_{50}}{y} = \left(\frac{K}{S_s - 1}\right)\left[\frac{V^2}{gy}\right]$$

Where:

D_{50} = median stone diameter (ft)
V = characteristic average velocity in the contracted section (ft/s)
Ss = specific gravity of rock riprap (2.65)
g = gravitational acceleration (32.2 ft/s^2)
y = depth of flow in the contracted bridge opening (ft)
K = 0.89 for a spill-through abutment and 1.02 for a vertical wall abutment

The D_{50} is a function of velocity and depth. The average channel flow velocities and flow depths during the design flow from the hydraulic analysis were used to calculate D_{50} of the RSP to protect the embankments at the bridge abutments. The D_{50} for the RSP was calculated immediately upstream, at the upstream face, at the downstream face, and immediately downstream of the bridge. The minimum RSP class for the existing bridge abutments calculated in accordance with the HEC-23 method is Class III. However, Class IV RSP is recommended based on engineering judgment. Per the HDM, Class IV RSP at the Project site requires a Class 8 RSP geotextile filter. The minimum RSP layer thickness is 2.5 ft, and detailed RSP calculations are in Appendix D.

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6 CONSTRUCTION SEASON FLOW

The purpose of the construction season flow rate study is to establish the relationship between risk and flow rates, to be used by the contractor to develop temporary diversion system design for the construction of the proposed Project.

The Project is located on Stevenson Bridge Road over Putah Creek. The Project's watershed was delineated based on USGS StreamStats, and the watershed at the Project site is approximately 644 sq. mi (see Section 2.2).

The USGS stream gage nearest to the Project location (USGS Gage 11454000) is located approximately 16 mi west of the Project site along Putah Creek (see Section 3.2.1). The watershed area of Putah Creek at this gaging station is approximately 574 sq. mi. The specifications of this gaging station are summarized in Table 7.

Table 7. USGS 11454000 Specifications

Location	38°30'55" North (NAD27)
	122°04'51" West (NAD27)
Watershed Area (sq. mi)	574
Record Time for Discharge in 15-min interval*	Begin: October 01, 1987
	End: December 29, 2017

Source: USGS 2017

* This gaging station is active. The data was last accessed in December 29, 2017.

The CVFPB has designated a non-permissible work period from November 1 through April 15 for Putah Creek. The typical summer period ranges from June to October, which is within the CVFPB's permissible work period for Putah Creek. There may be other seasonal work restrictions from other agencies or permit requirements.

Even though the gaging station is located upstream of the Project site, inflows or outflows are not taken into the consideration when determining the construction flow rate because the typical summer construction period ranges from June to October, which assumes little to no precipitation. The statistical analysis results for the construction season flow will be the same for the gaging station and Project site.

This USGS gaging station recorded the discharge rate of Putah Creek every 15-min starting from October 1, 1987. The gaging station data for the construction period from June 1 to October 15 were extracted for the analysis, and the minimum, average, and maximum peak flows recorded were calculated based on the extracted data from 1988 to 2017. The contractor may elect to work later in the season when flows are lower with the appropriate diversion system to move flows away from the necessary work area. The minimum, average, and maximum peak flows are summarized in Appendix E. Figure 13 shows the monthly minimum, average, and maximum discharges at gaging station 11454000.

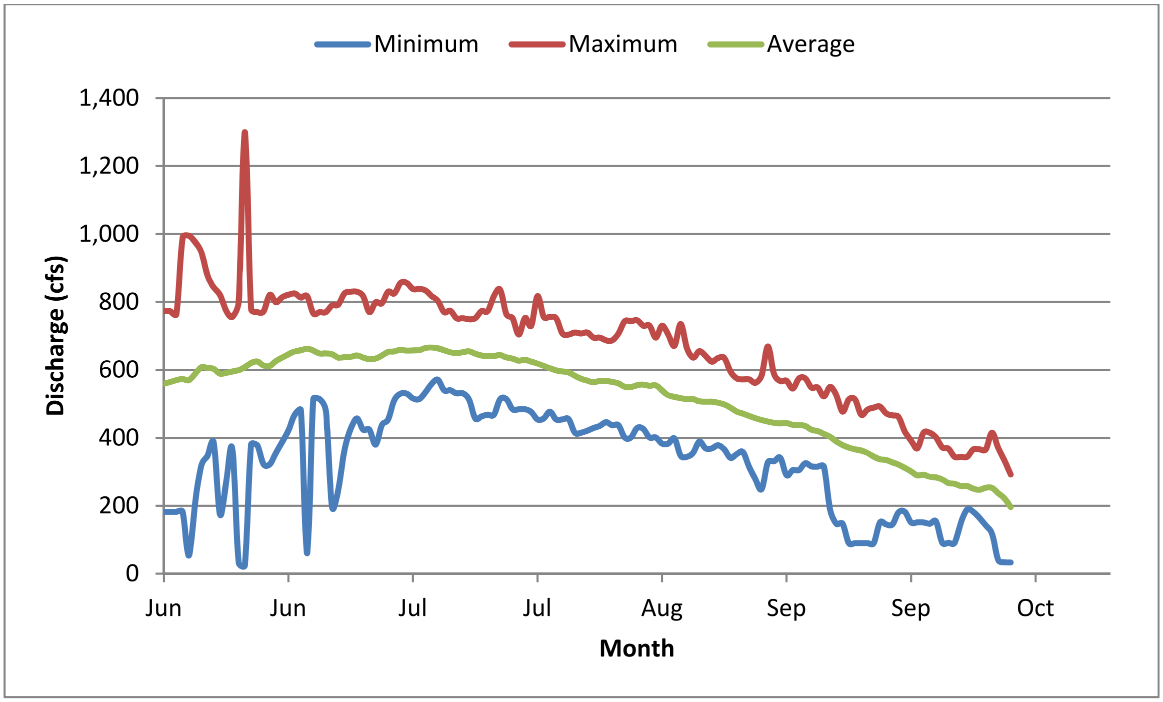

Figure 13. Discharges at gage 11454000 between 1988 and 2017

Source: USGS Gaging Station 11454000

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Appendix A HEC-RAS Calculations for the Existing Bridge

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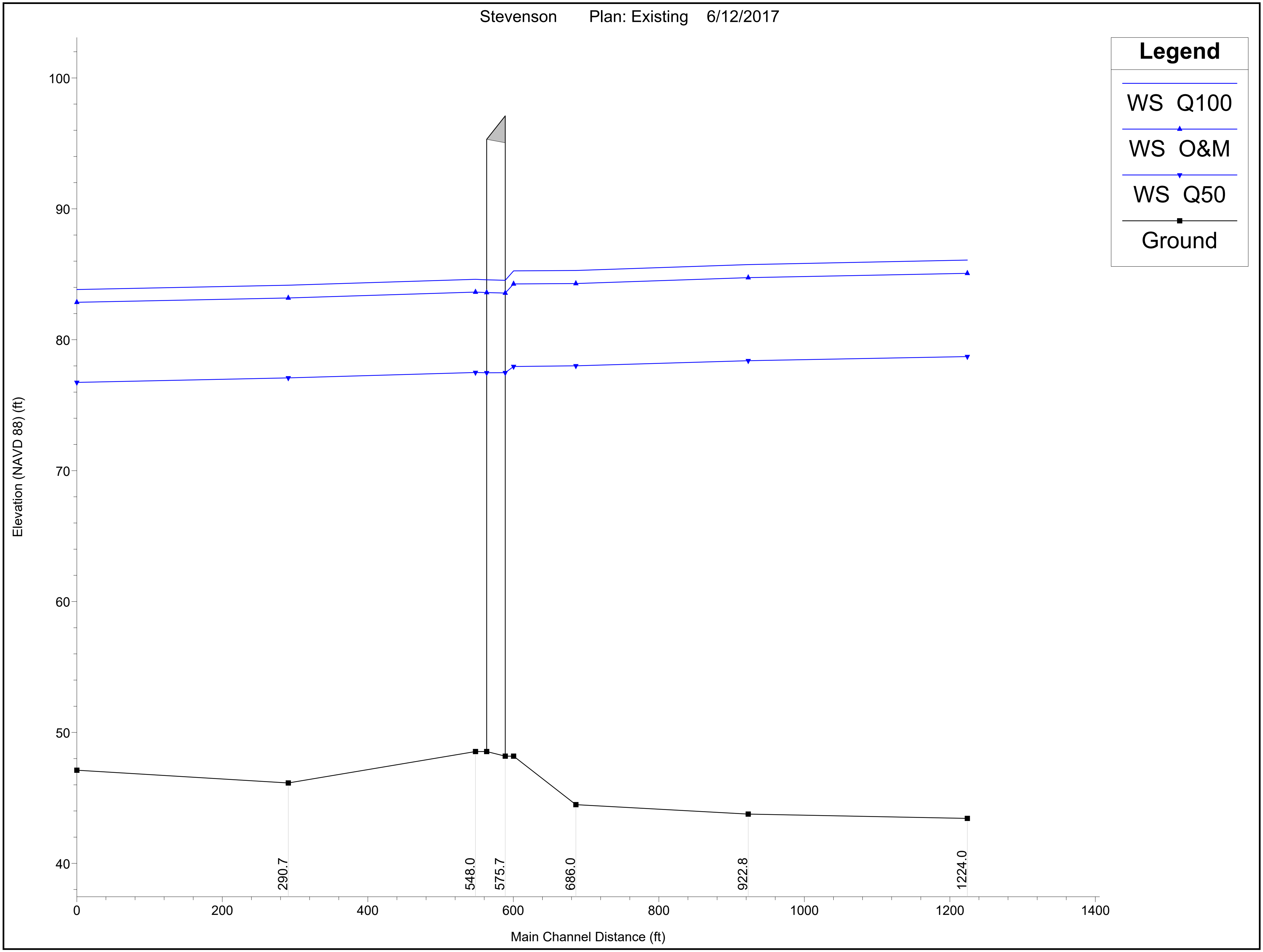
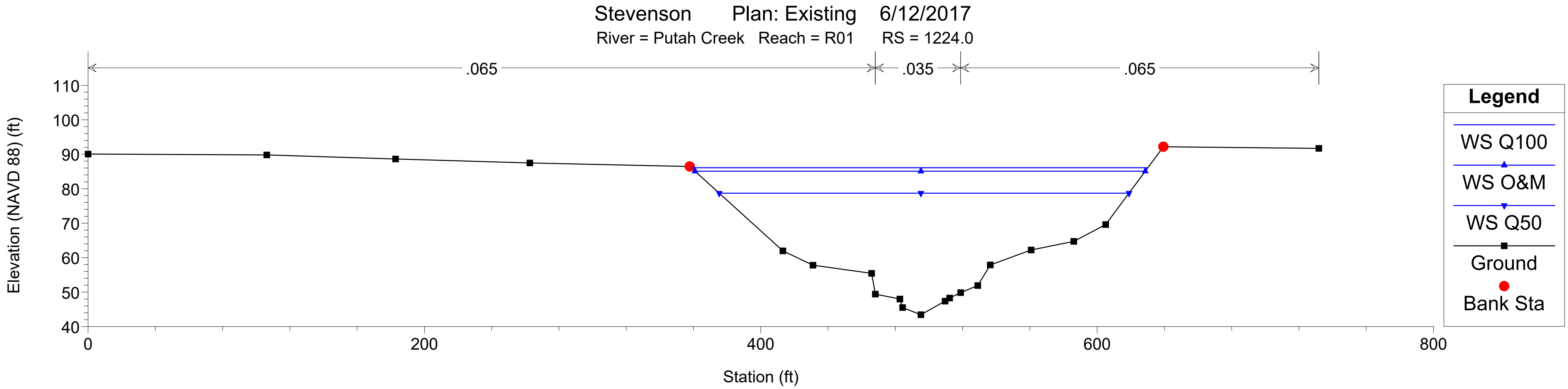
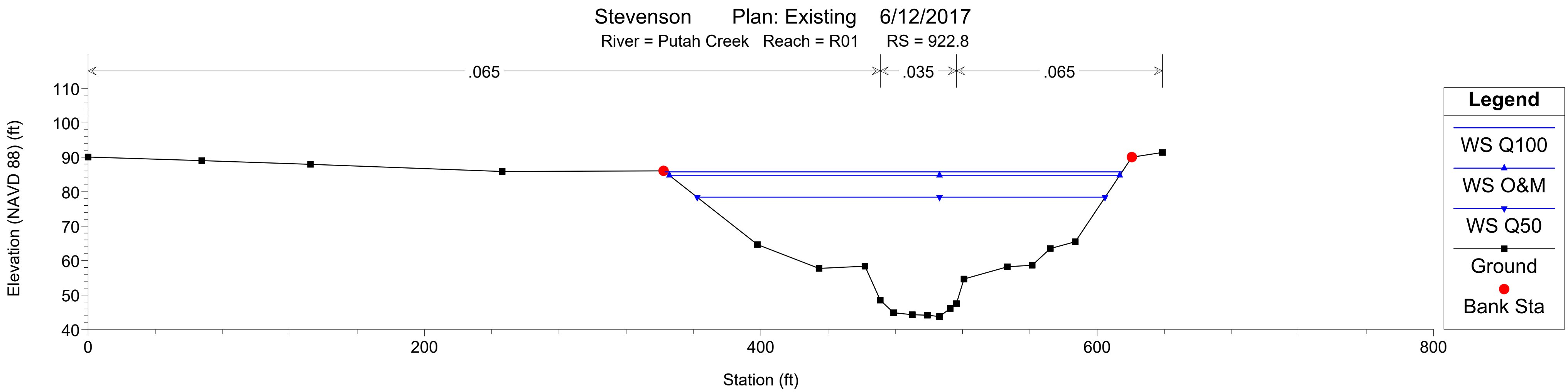


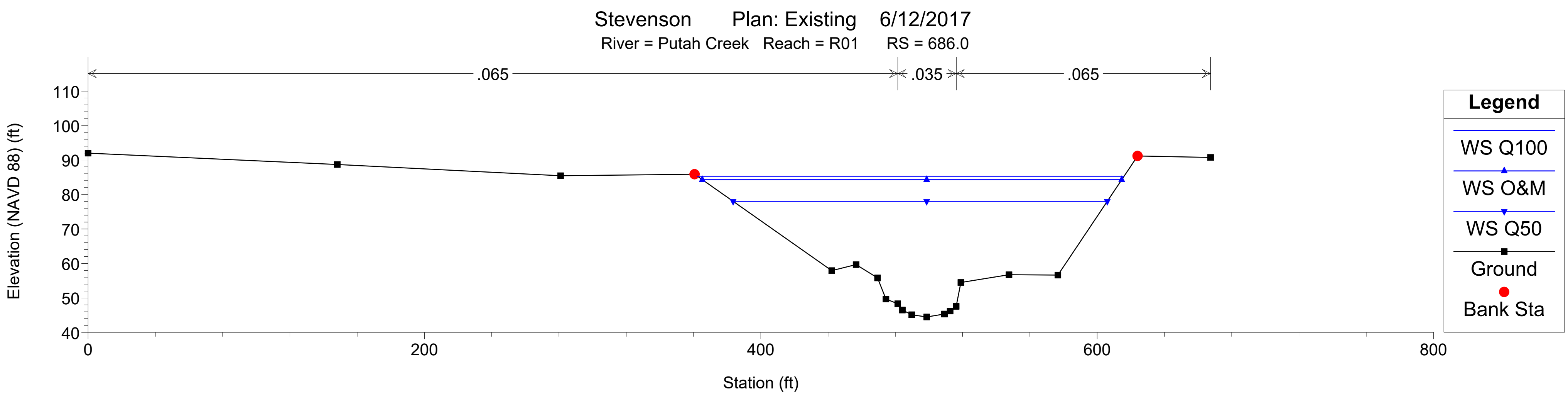
1 in Horiz. = 162 ft 1 in Vert. = 9 ft

HEC-RAS Plan: Existing   River: Putah Creek   Reach: R01

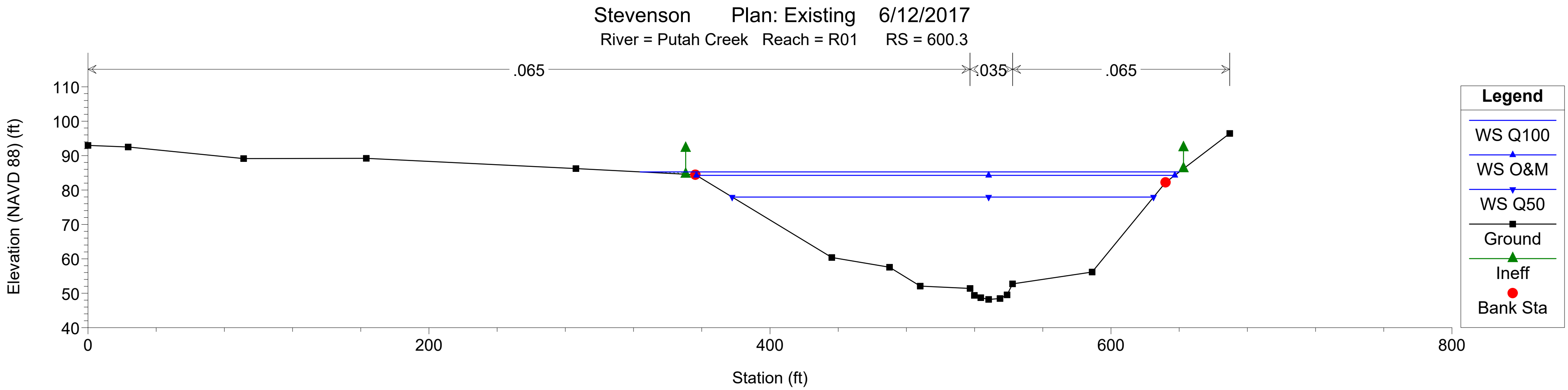
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Hydr Depth	Hydr Depth C	Length Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)	(ft)
R01	1224.0	Q100	42600.00	43.44	86.08		86.73	0.001079	6.47	6581.95	271.53	0.23	24.24	24.24	301.17
R01	1224.0	O&M	40000.00	43.44	85.08		85.70	0.001070	6.34	6310.37	267.72	0.23	23.57	23.57	301.17
R01	1224.0	Q50	25500.00	43.44	78.72		79.18	0.001010	5.44	4684.18	243.67	0.22	19.22	19.22	301.17
R01	922.8	Q100	42600.00	43.76	85.74		86.40	0.001129	6.51	6548.74	271.71	0.23	24.10	24.10	236.85
R01	922.8	O&M	40000.00	43.76	84.74		85.37	0.001120	6.37	6277.83	267.71	0.23	23.45	23.45	236.85
R01	922.8	Q50	25500.00	43.76	78.40		78.87	0.001054	5.47	4660.82	242.49	0.22	19.22	19.22	236.85
R01	686.0	Q100	42600.00	44.48	85.29		86.08	0.001460	7.14	5968.96	253.72	0.26	23.53	23.53	85.70
R01	686.0	O&M	40000.00	44.48	84.30		85.06	0.001448	7.00	5718.06	249.44	0.26	22.92	22.92	85.70
R01	686.0	Q50	25500.00	44.48	78.01		78.57	0.001351	6.02	4235.01	222.47	0.24	19.04	19.04	85.70
R01	600.3	Q100	42600.00	48.19	85.26	67.45	85.93	0.001216	6.57	6493.45	316.23	0.24	22.43	23.49	11.40
R01	600.3	O&M	40000.00	48.19	84.26	66.92	84.91	0.001237	6.45	6206.06	280.63	0.24	22.11	22.53	11.40
R01	600.3	Q50	25500.00	48.19	77.95	63.67	78.44	0.001211	5.61	4545.72	247.17	0.23	18.39	18.39	11.40
R01	575.7 BR U	Q100	42600.00	48.19	84.54	69.28	85.75	0.005244	8.83	4832.99	215.22	0.32	22.46	23.54	25.50
R01	575.7 BR U	O&M	40000.00	48.19	83.57	68.68	84.73	0.005127	8.64	4630.35	205.69	0.32	22.51	22.90	25.50
R01	575.7 BR U	Q50	25500.00	48.19	77.48	65.23	78.31	0.004014	7.33	3477.40	175.59	0.29	19.80	19.80	25.50
R01	575.7 BR D	Q100	42600.00	48.54	84.58	69.09	85.52	0.002757	7.80	5470.97	234.79	0.23	23.30	23.42	15.41
R01	575.7 BR D	O&M	40000.00	48.54	83.60	68.53	84.51	0.002746	7.64	5241.98	234.79	0.23	22.33	22.45	15.41
R01	575.7 BR D	Q50	25500.00	48.54	77.47	65.09	78.16	0.002628	6.63	3845.18	219.81	0.28	17.49	17.50	15.41
R01	548.0	Q100	42600.00	48.54	84.61		85.42	0.001425	7.21	5914.23	261.34	0.26	23.32	23.43	257.25
R01	548.0	O&M	40000.00	48.54	83.64		84.41	0.001429	7.07	5667.20	257.92	0.26	22.53	22.65	257.25
R01	548.0	Q50	25500.00	48.54	77.49		78.08	0.001486	6.12	4164.47	236.48	0.26	17.61	17.62	257.25
R01	290.7	Q100	42600.00	46.14	84.16		85.03	0.001471	7.48	5697.54	249.70	0.28	22.82	22.82	290.72
R01	290.7	O&M	40000.00	46.14	83.19		84.02	0.001457	7.33	5456.13	245.44	0.27	22.23	22.23	290.72
R01	290.7	Q50	25500.00	46.14	77.08		77.70	0.001336	6.31	4039.51	218.77	0.26	18.46	18.46	290.72
R01	0.0	Q100	42600.00	47.11	83.84	67.14	84.58	0.001401	6.90	6171.65	274.20	0.26	22.51	22.51	
R01	0.0	O&M	40000.00	47.11	82.86	66.59	83.57	0.001401	6.77	5905.49	270.47	0.26	21.83	21.83	
R01	0.0	Q50	25500.00	47.11	76.74	63.29	77.28	0.001400	5.90	4322.58	247.12	0.25	17.49	17.49	

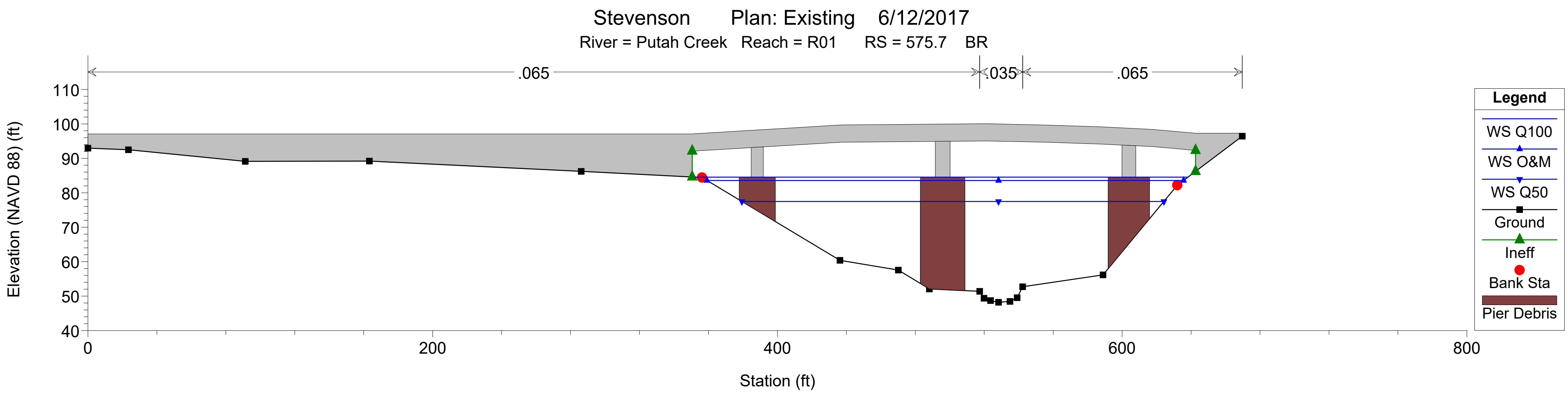


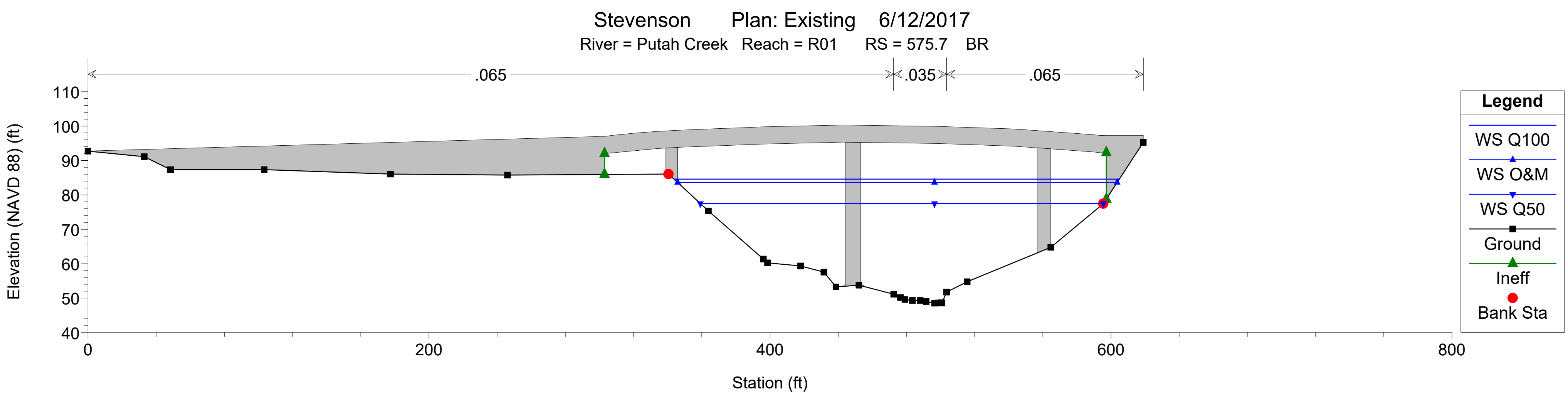




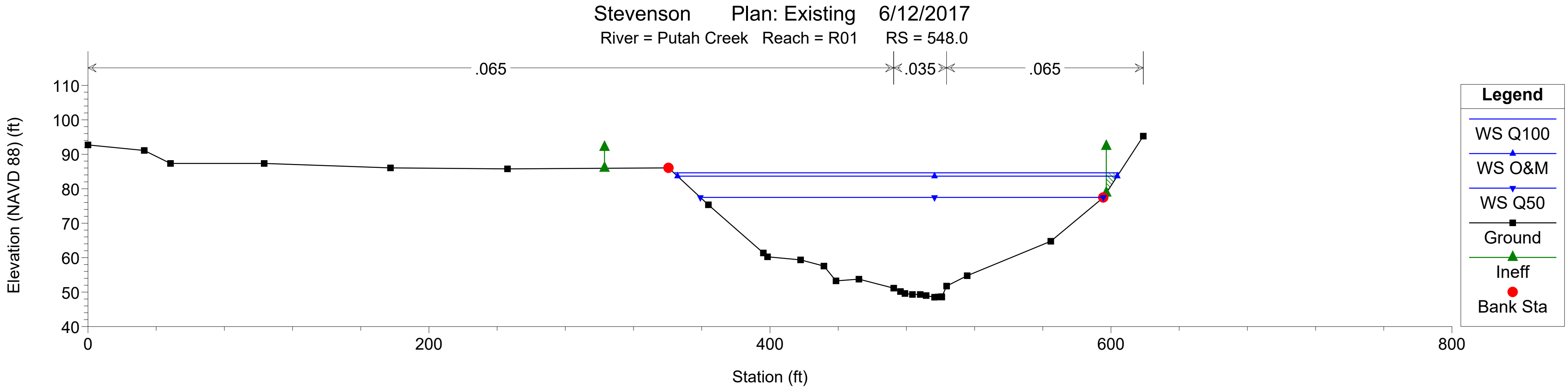
1 in Horiz. = 88 ft 1 in Vert. = 43 ft

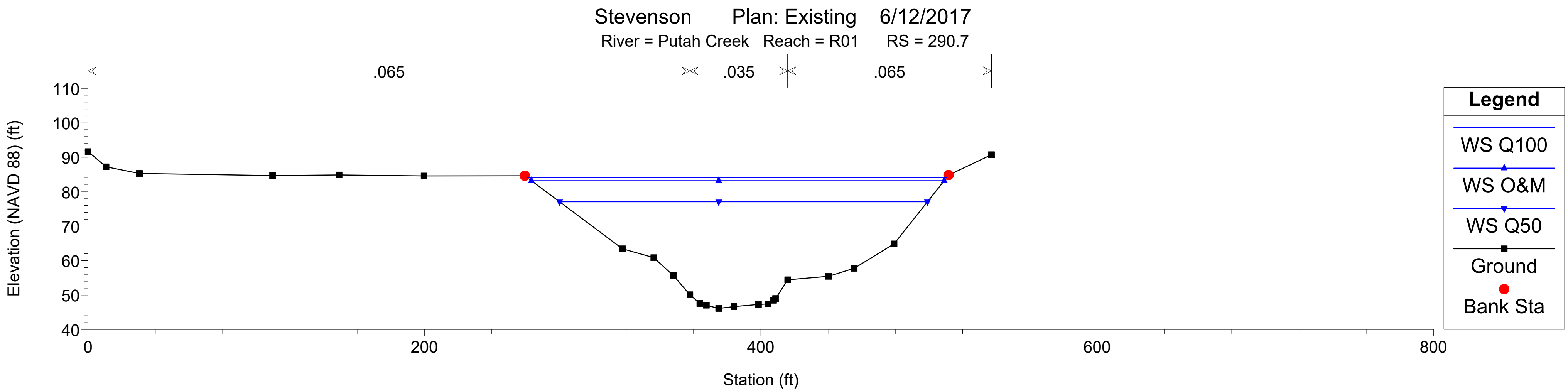


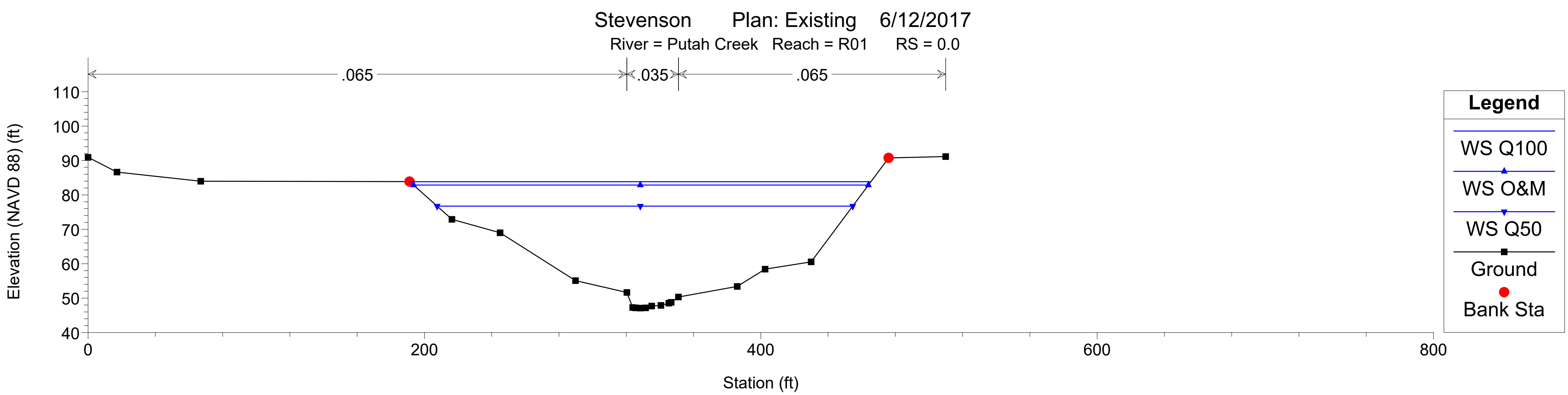




1 in Horiz. = 88 ft 1 in Vert. = 43 ft







1 in Horiz. = 88 ft 1 in Vert. = 43 ft

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Appendix B HEC-RAS Calculations for the Proposed Bridge

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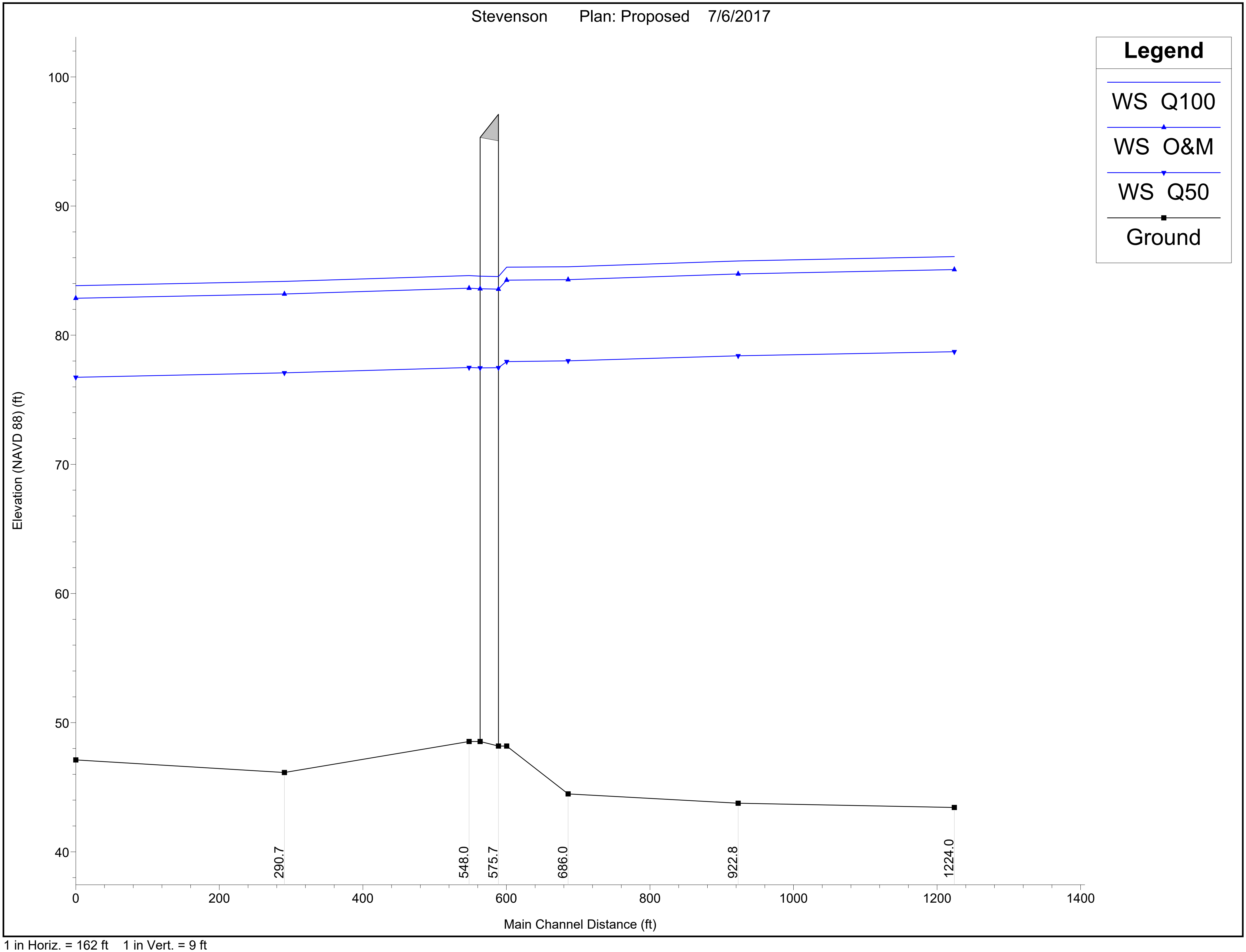
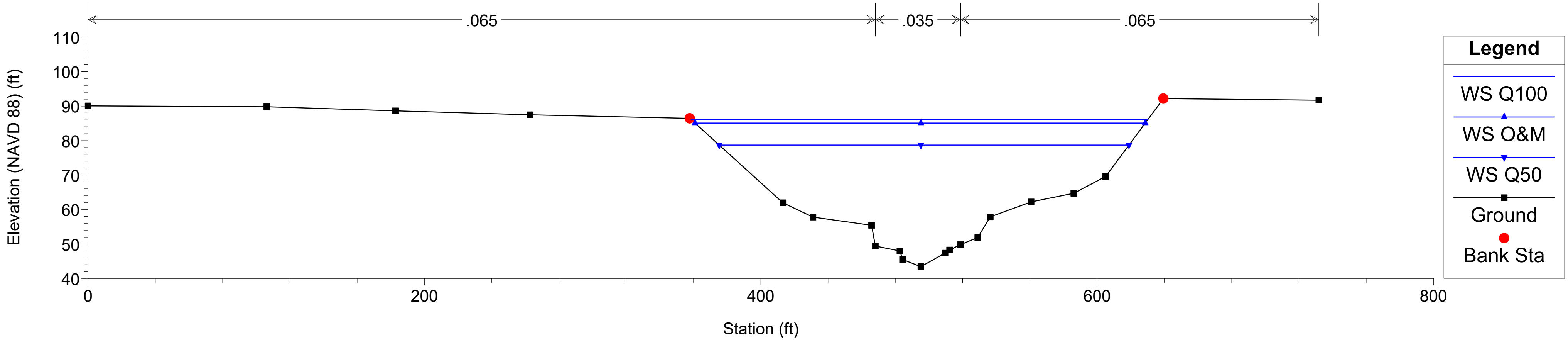


Stevenson Plan: Proposed 7/6/2017
Legend
WS Q100
WS O&M
WS Q50
Ground
Elevation (NAVD 88) (ft)
100
90
80
70
60
50
40
290.7
548.0
575.7
686.0
922.8
1224.0
0
200
400
600
800
1000
1200
1400
Main Channel Distance (ft)
1 in Horiz. = 162 ft 1 in Vert. = 9 ft

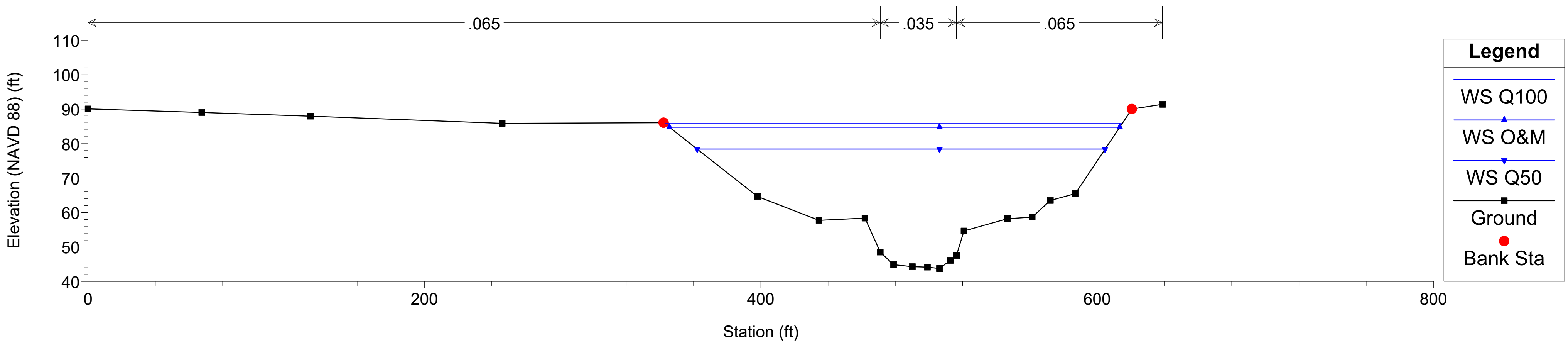
HEC-RAS Plan: Proposed River: Putah Creek Reach: R01

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Hydr Depth	Hydr Depth C	Length Chnl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)	(ft)	(ft)
R01	1224.0	Q100	42600.00	43.44	86.08		86.74	0.001078	6.47	6582.38	271.54	0.23	24.24	24.24	301.17
R01	1224.0	O&M	40000.00	43.44	85.08		85.70	0.001070	6.34	6310.80	267.73	0.23	23.57	23.57	301.17
R01	1224.0	Q50	25500.00	43.44	78.72		79.18	0.001010	5.44	4684.61	243.68	0.22	19.22	19.22	301.17
R01	922.8	Q100	42600.00	43.76	85.74		86.40	0.001129	6.50	6549.19	271.72	0.23	24.10	24.10	236.85
R01	922.8	O&M	40000.00	43.76	84.74		85.37	0.001120	6.37	6278.27	267.72	0.23	23.45	23.45	236.85
R01	922.8	Q50	25500.00	43.76	78.40		78.87	0.001053	5.47	4661.27	242.49	0.22	19.22	19.22	236.85
R01	686.0	Q100	42600.00	44.48	85.29		86.09	0.001460	7.14	5969.40	253.73	0.26	23.53	23.53	85.70
R01	686.0	O&M	40000.00	44.48	84.30		85.06	0.001448	6.99	5718.50	249.45	0.26	22.92	22.92	85.70
R01	686.0	Q50	25500.00	44.48	78.01		78.58	0.001350	6.02	4235.44	222.48	0.24	19.04	19.04	85.70
R01	600.3	Q100	42600.00	48.19	85.26	67.45	85.94	0.001216	6.57	6493.96	316.30	0.24	22.43	23.49	11.40
R01	600.3	O&M	40000.00	48.19	84.26	66.92	84.91	0.001237	6.45	6206.56	280.64	0.24	22.12	22.53	11.40
R01	600.3	Q50	25500.00	48.19	77.95	63.67	78.44	0.001210	5.61	4546.20	247.18	0.23	18.39	18.39	11.40
R01	575.7 BR U	Q100	42600.00	48.19	84.54	69.29	85.75	0.005229	8.83	4831.82	215.25	0.32	22.45	23.54	25.50
R01	575.7 BR U	O&M	40000.00	48.19	83.57	68.69	84.73	0.005112	8.64	4629.17	205.69	0.32	22.51	22.89	25.50
R01	575.7 BR U	Q50	25500.00	48.19	77.48	65.23	78.32	0.004000	7.34	3476.23	175.59	0.29	19.80	19.80	25.50
R01	575.7 BR D	Q100	42600.00	48.54	84.57	69.26	85.53	0.002811	7.84	5442.11	234.79	0.23	23.18	23.30	15.41
R01	575.7 BR D	O&M	40000.00	48.54	83.60	68.71	84.51	0.002802	7.68	5213.12	234.79	0.23	22.20	22.33	15.41
R01	575.7 BR D	Q50	25500.00	48.54	77.47	65.28	78.16	0.002699	6.68	3816.39	219.79	0.28	17.36	17.37	15.41
R01	548.0	Q100	42600.00	48.54	84.61		85.42	0.001425	7.21	5914.23	261.34	0.26	23.32	23.43	257.25
R01	548.0	O&M	40000.00	48.54	83.64		84.41	0.001429	7.07	5667.20	257.92	0.26	22.53	22.65	257.25
R01	548.0	Q50	25500.00	48.54	77.49		78.08	0.001486	6.12	4164.47	236.48	0.26	17.61	17.62	257.25
R01	290.7	Q100	42600.00	46.14	84.16		85.03	0.001471	7.48	5697.54	249.70	0.28	22.82	22.82	290.72
R01	290.7	O&M	40000.00	46.14	83.19		84.02	0.001457	7.33	5456.12	245.44	0.27	22.23	22.23	290.72
R01	290.7	Q50	25500.00	46.14	77.08		77.70	0.001336	6.31	4039.51	218.77	0.26	18.46	18.46	290.72
R01	0.0	Q100	42600.00	47.11	83.84	67.14	84.58	0.001401	6.90	6171.65	274.20	0.26	22.51	22.51	
R01	0.0	O&M	40000.00	47.11	82.86	66.59	83.57	0.001401	6.77	5905.49	270.47	0.26	21.83	21.83	
R01	0.0	Q50	25500.00	47.11	76.74	63.29	77.28	0.001400	5.90	4322.58	247.12	0.25	17.49	17.49	

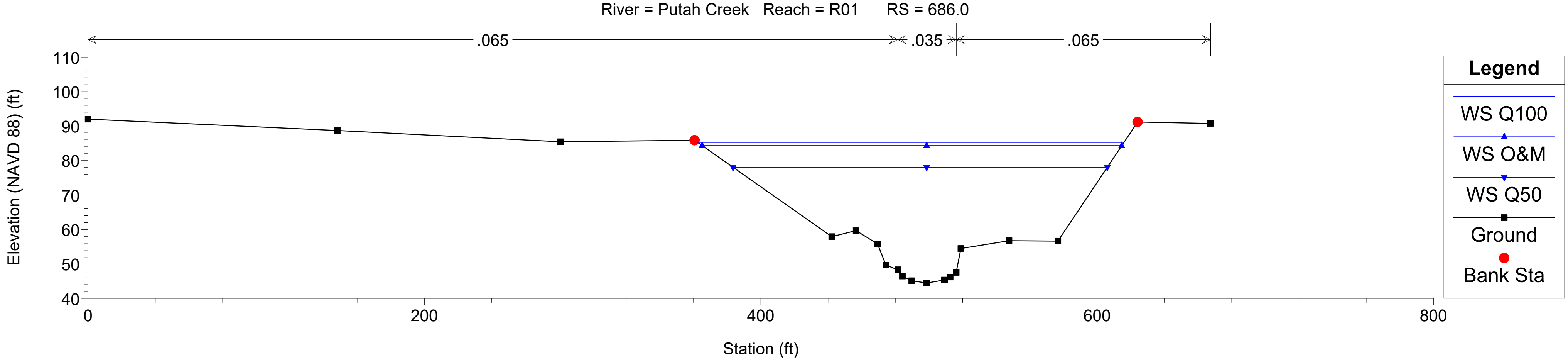
Stevenson Plan: Proposed 7/6/2017
River = Putah Creek Reach = R01 RS = 1224.0



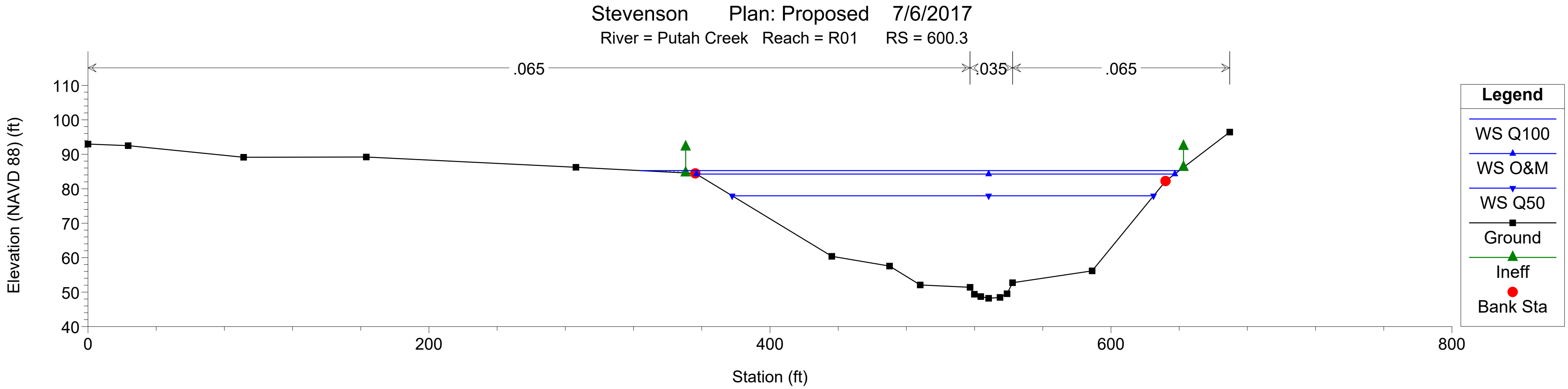
Stevenson Plan: Proposed 7/6/2017
River = Putah Creek Reach = R01 RS = 922.8

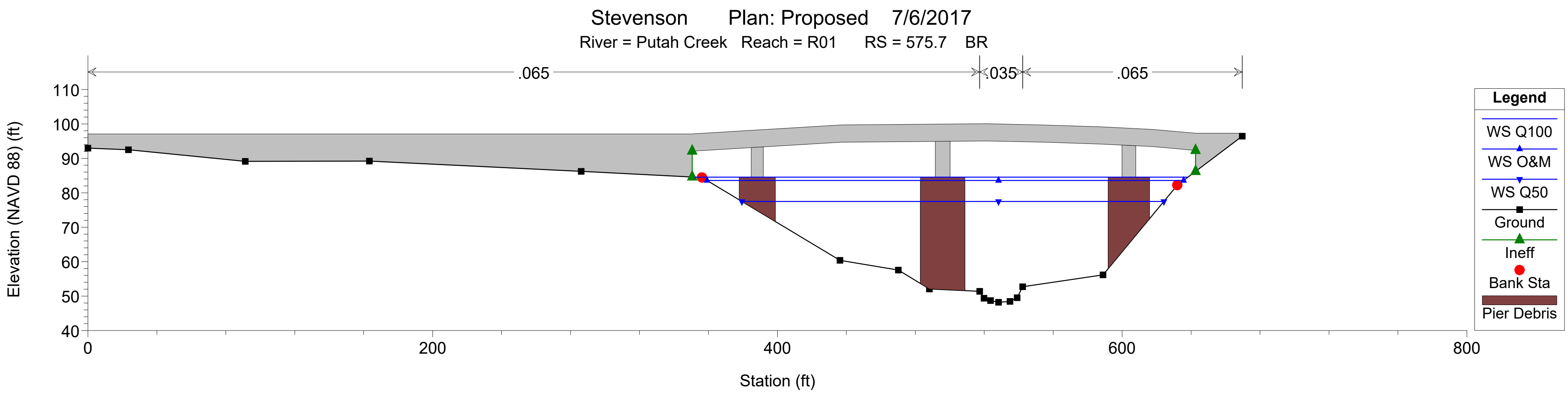


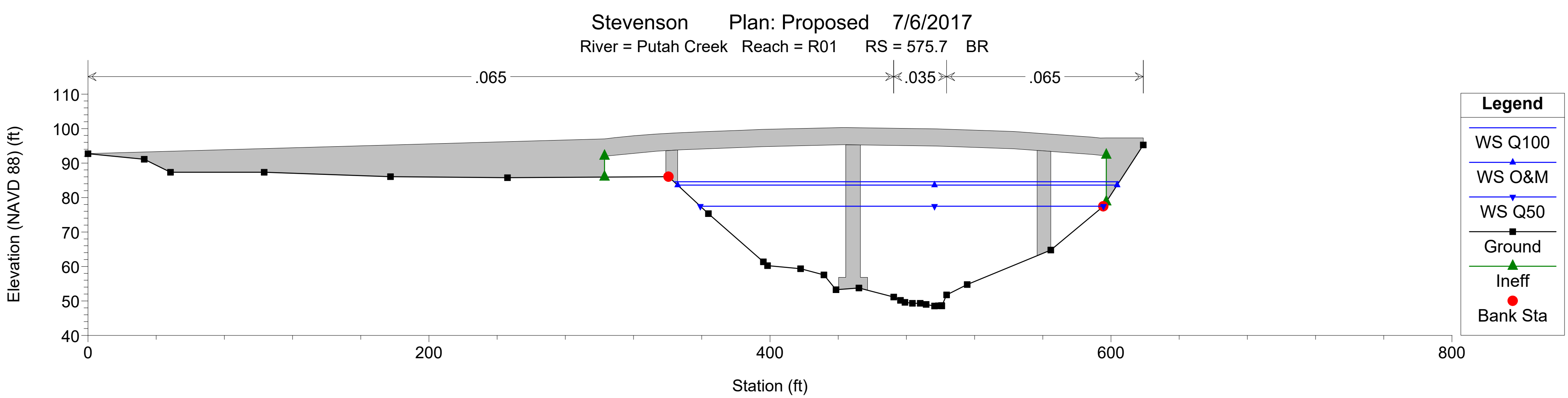
Stevenson Plan: Proposed 7/6/2017
River = Putah Creek Reach = R01 RS = 686.0



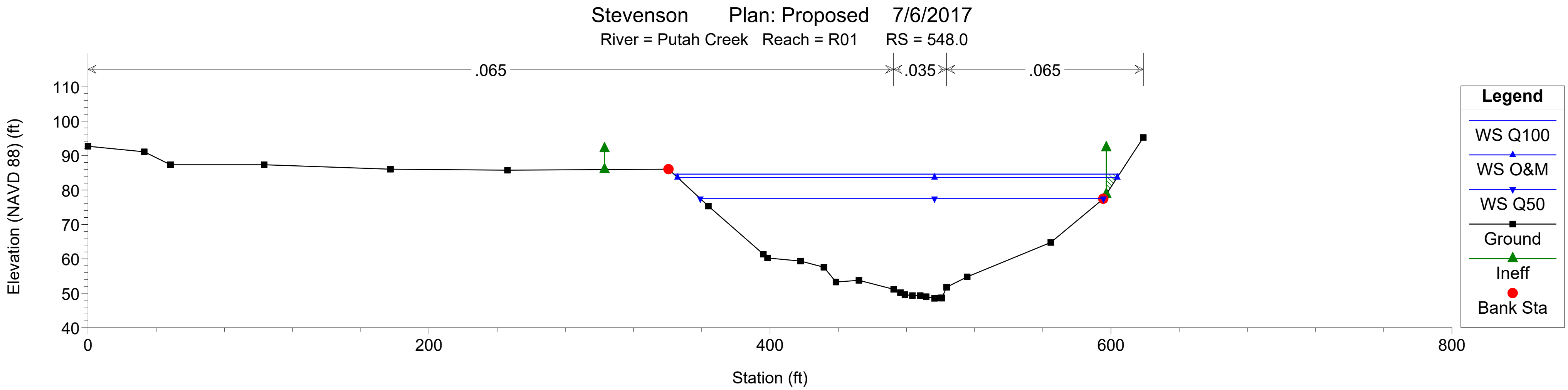
1 in Horiz. = 88 ft 1 in Vert. = 43 ft

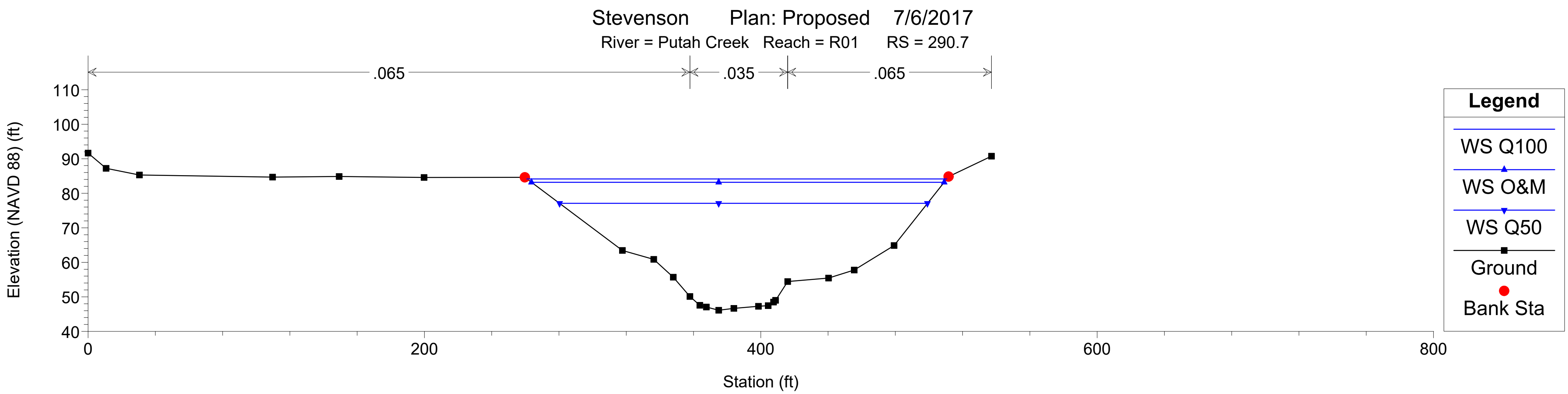


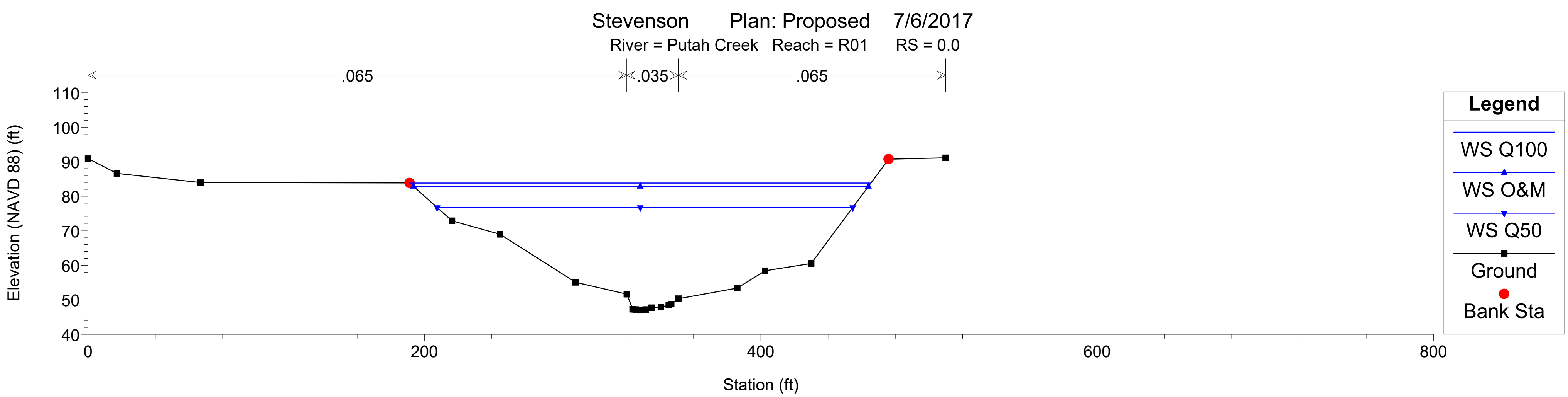




1 in Horiz. = 88 ft 1 in Vert. = 43 ft







1 in Horiz. = 88 ft 1 in Vert. = 43 ft

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Appendix C Scour Calculations

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Stevenson Rd Bridge
Solano County, California
Local Scour at Piers - Cohesive

100-year Flow
Calculation guideline from HEC-18 5th Edition
Input from HEC-RAS for Existing Condition with Log-Pearson Flow
Equation from FHWA HEC-18 5th Edition: Page 7.38, Page 204 / 340 , Section 7.12 Pier Scour In Cohesive Materials
Equation 7.35:

$$y_s = 2.2 K_1 K_2 a^{0.65} \left(\frac{2.6 V_1 - V_c}{\sqrt{g}} \right)^{0.7}$$

Variable	Value				Description
Pier Number (Plan)	2	3	4		
Pier Number (HEC-RAS)	3	2	1		
L	25.5	25.5	8.7	ft	Pier length
	8.6	8.6	7.6	ft	Top Pier Width
	17.0	17.0	17.0		Cap Width
	7	7	7	ft	CIDH Pile Width
a	**9.7**	**9.0**	**9.3**	ft	Weighted Pier width
L/a	2.6	2.8	0.9		If L/a is larger than 12, then use 12 as a maximum
Θ	0	0	0	degrees	Angle of attack of flow
	Weighted	Weighted	Weighted		Pier shape
K1	1.09	1.07	1.08		Correction factor for pier shape
K2	1.0	1.0	1.0		Correction factor for angle of attack
V1	4.4	6.7	3.2	ft/s	Approach velocity
Vc	0.3	0.3	0.3	m/s	From Figure 4.7:
Vc	1.0	1.0	1.0	ft/s	using an erosion rate of 0.1 mm/hr
g	32.2	32.2	32.2	ft/s^2	and based on ML
ys	**16.1**	**20.6**	**12.1**	**ft**	**Pier Scour**

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Appendix D RSP Calculations

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P16044 Stevenson Bridge

Solano County, California

Streambank Rock Slope Protection

Calculation guideline from Caltrans Highway Design Manual

Input from HEC-RAS for Proposed Condition
100-year Flow

Input

Location along stream:	Upstream	Upstream Face	Downstream Face	Downstream		
V_{avg}	6.0	7.1	6.6	6.1	ft/s	Average channel velocity
g	32.2	32.2	32.2	32.2	ft/s^2	Acceleration due to gravity
Depth based on	Average	Average	Average	Average		Average Local
y	19.0	20.0	17.6	17.6	ft	Local depth of flow (toe of slope is typically used for bank revetment applications; average channel depth can be used)
S_f	1.1	1.1	1.1	1.1		Safety factor (typically = 1.1)
C_s	0.3	0.3	0.3	0.3		Stability coefficient (for blanket thickness $1.5d_{50}$ or d_{100}, whichever is greater) = 0.30 for angular rock
Cross section location:	Straight channel	Straight channel	Straight channel	Straight channel		Straight cha Inside of be Outside of b Downstrean End of dike
C_v	1.00	1.00	1.00	1.00		Velocity distribution coefficient (1.0 for straight channels or the inside of bends;
For outside of bends, need R_c and W:						
R_c	100.0	100.0	100.0	100.0	ft	Centerline radius of curvature of channel bend
W	1.0	1.0	1.0	1.0	ft	Width of water surface at upstream end of channel bend
C_t	1.0	1.0	1.0	1.0		Blanket thickness coefficient = 1.0
S_g	2.65	2.65	2.65	2.65		Specific gravity of stone (2.5 minimum)
Type of channel:	Natural	Natural	Natural	Natural		Natural Trapezoidal
V_{des}	6.0	7.1	6.6	6.1	ft/s	Characteristic velocity for design; depth-averaged velocity at a point 20% upslope from the toe of revetment
K_1	0.72	0.72	0.72	0.72		Side slope correction factor
θ	33.7	33.7	33.7	33.7	degrees	Bank angle
SS	1.5	1.5	1.5	1.5		Side slope (horizontal to 1 vertical); 1.5 or flatter.
D_{30}	0.1	0.2	0.2	0.2	ft	Particle size for which 30% is finer by weight
D_{50}	0.2	0.3	0.2	0.2	ft	Particle size for which 50% is finer by weight
D50	2.2	3.2	2.7	2.3	in	Particle size for which 50% is finer by weight
	Class I	**Class I**	**Class I**	**Class I**	**RSP Class**	**[Select the next larger size class.]**

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P16044 Stevenson Bridge
Solano County, California
Rock Slope Protection Calculations for Abutments
Calculation guideline from HEC-23 3rd Edition

Input from HEC-RAS for Proposed Condition

100-year Flow

Location	Upstream	Upstream Face	Downstream Face	Downstream	
V	6.0	7.1	6.6	6.1	ft/s
g	32.2	32.2	32.2	32.2	ft/s^2
y	19.0	20.0	17.6	17.6	ft
Fr	0.24	0.28	0.28	0.26	
Equation	**Isbash**	**Isbash**	**Isbash**	**Isbash**	

For Froude Numbers $(V/(gy)^{1/2})$<=0.80, Isbash relationship (Equation 14.1)

$$D_{50} = \frac{yK}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

y	19.0	20.0	17.6	17.6	depth of flow in the contracted bridge opening, ft
K	1.02	1.02	1.02	1.02	1.02 for vertical wall abutment, 0.89 or for spill-through abutment
S_s	2.65	2.65	2.65	2.65	specific gravity of rock
V	6.0	7.1	6.6	6.1	average velocity in contracted section, ft/s
g	32.2	32.2	32.2	32.2	gravitational acceleration, ft/s^2
D_{50}	0.7	1.0	0.8	0.7	median stone diameter, ft
D_{50}	8.4	11.5	9.9	8.6	median stone diameter, inches
	Class II	Class III	Class III	Class II	rock class

Appendix E Construction Summer Flow

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Average values for each day for 29 - 30 years of record in ft3/s
Years of Data: 1988-2017 for Calculation Period: June 1 through October 15

Day of month	Discharge (cubic feet per second)				
	Jun	Jul	Aug	Sep	Oct
1	559	638	612	477	291
2	564	642	605	471	285
3	570	636	598	464	284
4	573	632	595	458	277
5	570	633	592	452	267
6	588	642	583	448	265
7	608	653	574	444	258
8	605	654	568	443	258
9	603	659	564	443	250
10	589	657	568	438	247
11	591	657	567	437	253
12	596	658	565	435	252
13	599	664	560	424	237
14	608	665	550	420	222
15	620	664	549	412	196
16	625	658	555	403	
17	613	652	557	389	
18	612	649	553	379	
19	626	652	554	371	
20	636	655	540	366	
21	646	648	525	362	
22	654	642	521	356	
23	658	641	517	344	
24	662	641	514	337	
25	657	644	514	335	
26	648	637	507	328	
27	648	633	506	321	
28	646	627	506	311	
29	636	630	503	301	
30	637	624	498	289	
31		618	489		

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Maximum values for each day for 29 - 30 years of record in ft3/s
Years of Data: 1988-2017 for Calculation Period: June 1 through October 15

Day of month	Discharge (cubic feet per second)				
	Jun	Jul	Aug	Sep	Oct
1	773	830	756	575	415
2	773	830	756	572	415
3	764	817	752	572	401
4	990	770	707	562	372
5	995	799	704	584	368
6	976	796	710	669	344
7	947	830	707	587	344
8	878	825	710	567	344
9	843	856	695	568	366
10	821	856	695	545	366
11	774	838	687	575	366
12	756	838	687	575	415
13	799	834	707	548	370
14	1,300	817	743	548	333
15	779	803	743	522	292
16	770	770	746	550	
17	772	774	730	525	
18	821	752	730	477	
19	799	752	695	513	
20	814	749	730	513	
21	821	752	704	468	
22	825	773	671	483	
23	813	773	735	489	
24	817	817	664	492	
25	766	836	636	472	
26	770	764	655	466	
27	770	752	641	461	
28	790	704	624	418	
29	792	753	635	392	
30	825	730	635	369	
31		817	595		

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Minimum values for each day for 29 - 30 years of record in ft3/s
Years of Data: 1988-2017 for Calculation Period: June 1 through October 15

Day of month	Discharge (cubic feet per second)				
	Jun	Jul	Aug	Sep	Oct
1	182	426	457	352	151
2	182	457	477	358	147
3	182	424	454	314	154
4	182	424	454	278	91
5	53	380	455	249	91
6	205	437	415	328	91
7	318	454	415	331	150
8	347	511	421	341	189
9	386	530	429	290	182
10	175	530	435	305	164
11	270	516	446	305	142
12	366	514	437	325	118
13	33	534	437	316	40
14	25	559	401	315	34
15	380	571	401	315	33
16	378	540	429	188	
17	322	540	426	147	
18	322	531	401	147	
19	356	531	401	90	
20	388	513	383	90	
21	422	457	383	90	
22	468	463	398	90	
23	481	468	347	90	
24	60	468	344	151	
25	514	514	356	145	
26	514	514	389	145	
27	481	484	369	181	
28	197	484	369	182	
29	247	484	378	151	
30	364	476	366	151	
31		454	341		

Appendix G - Field Investigation Report

STEVENSON BRIDGE ROAD BRIDGE

BR NO 23C0092 - Field Investigation

March 31, 2017

Prepared for Quincy Engineering, Inc. and the County of Solano

Prepared by Alta Vista Solutions, Inc.

PROJECT INFORMATION

Field Investigation – Steven Bridge Road Bridge (Br. No. 23C0092)

SUBJECT

Risk Based Structural Assessment of Stevenson Road Bridge in Solano County.

BACKGROUND

Stevenson Bridge Road Bridge is a historical structure located in Winters, CA. Covered in brightly colored graffiti, it is locally known as Graffiti Bridge. It was built in 1923 and spans Putah Creek at the junction of Yolo County Road 95A and Solano County's Stevenson Bridge Road. The bridge consists of reinforced concrete T-beam approach spans and concrete tied arch main spans. The bridge structure is 296 feet long and 24 feet wide with two 40-foot approach spans and two 108-foot tied arch main spans. The substructure is supported on abutments with spread footings, two piers on timber piles and one pier on concrete piles. Carrying two lanes of two-way traffic, the structure is surrounded by farmland and experiences typically local residential traffic, farm vehicles and equipment, and bicyclists.

Figure 1 - Overhead view of Stevenson Bridge and surrounding area taken with unmanned aircraft system

The County of Solano, in conjunction with the County of Yolo and the California Department of Transportation (Caltrans), is proposing to rehabilitate and retrofit the bridge in accordance with FHWA guidelines under the Highway Bridge Program (HBP). This report summarizes the findings observed through various investigations on the structure.

Caltrans' bridge inspection report dated March 25, 2015 notes that the structure is functionally obsolete. Sour was compared with measurements in 2007 and reports 8 inches degradation in the channel at Pier 3, and 10 inches degradation at Pier 4. Cracks in girders at Spans 1 and 4 are reported to extend to the soffit. Additional cracks on girders at spans 2 and 3 are estimated at 20% of the length of the girders. The report also notes that transverse cracks at spans 1 and 4 appear to not have changed since 2009. Numerous spalls on the bridge should be patched and exposed rebar should be cleaned and painted to prevent further deterioration.

Previous studies conducted on the bridge are included in a Feasibility Study produced by TRC Imbsen in February 2007. This report evaluated the potential vulnerabilities of the structure and presented different rehabilitation and retrofit alternatives. These were compared with structure replacement. Factors taken into account include the historical aspect of the bridge, impact to local communities, and environmental considerations. Cost-wise, replacing the bridge would be comparable in magnitude with rehabilitation or retrofit. The report recommended a retrofit to be performed.

Figure 2 - Street view of Stevenson Bridge

DISCUSSION

Alta Vista Solutions, Inc. (Alta Vista) was hired to perform a field investigation of the current condition of the Stevenson Bridge Road Bridge structure and provide recommendations for repair strategies. The investigation used various tools, including image collection using an unmanned aircraft system (UAS), borescope inspection, ground penetrating radar (GPR) and performing strength tests on concrete cores extracted from the structure

Below is a summary of items discussed in this report to characterize the overall condition of the structure:

A. Overview of Field Activities
B. Visual Inspection
C. Borescope Observation
D. Ground Penetrating Radar (GPR) Scanning
E. Concrete Core Testing

INVESTIGATION

A. Field Activities

This bridge is composed of several structural elements. Select elements were inspected as part of this risk based assessment was subject to one or more inspection methods. Table 1 provides a summary of inspection and testing tools that were utilized for select elements of the bridge. Figure 3 shows the labeling convention used to identify each of the elements. On the west and east sides of the bridge, each element was numbered starting from the south end moving north. A drawing of the various work locations can be found in **Appendix 1**.

Table 1 - Inspection tools used for each element of the structure.

Component*	Visual Inspection and Imaging	Unmanned Aircraft System (UAS) Imaging	Borescope	Ground Penetrating Radar (GPR)	Concrete Compressive Strength
Bridge Deck	✓	✓	✓	✓	✓
Bridge Soffit	✓	✓			
Transverse Floor Beams	✓	✓			
Girders	✓	✓			✓
Arches	✓	✓	✓	✓	✓
Vertical Hangers	✓	✓	✓	✓	
Portal Bracing	✓	✓			
Pier Columns	✓	✓	✓	✓	

* Selected areas were investigated based on initial risk assessment

Field work incorporating each of the above mentioned activities was scheduled on various dates to allow for phasing of operations, including review and interpretation of data.

- August - Visual inspection and high definition photos
- September - UAS imaging operation
- November - Risk based field investigation
- December - Concrete testing and data analysis

The bridge elements are identified as shown in the diagrams below.

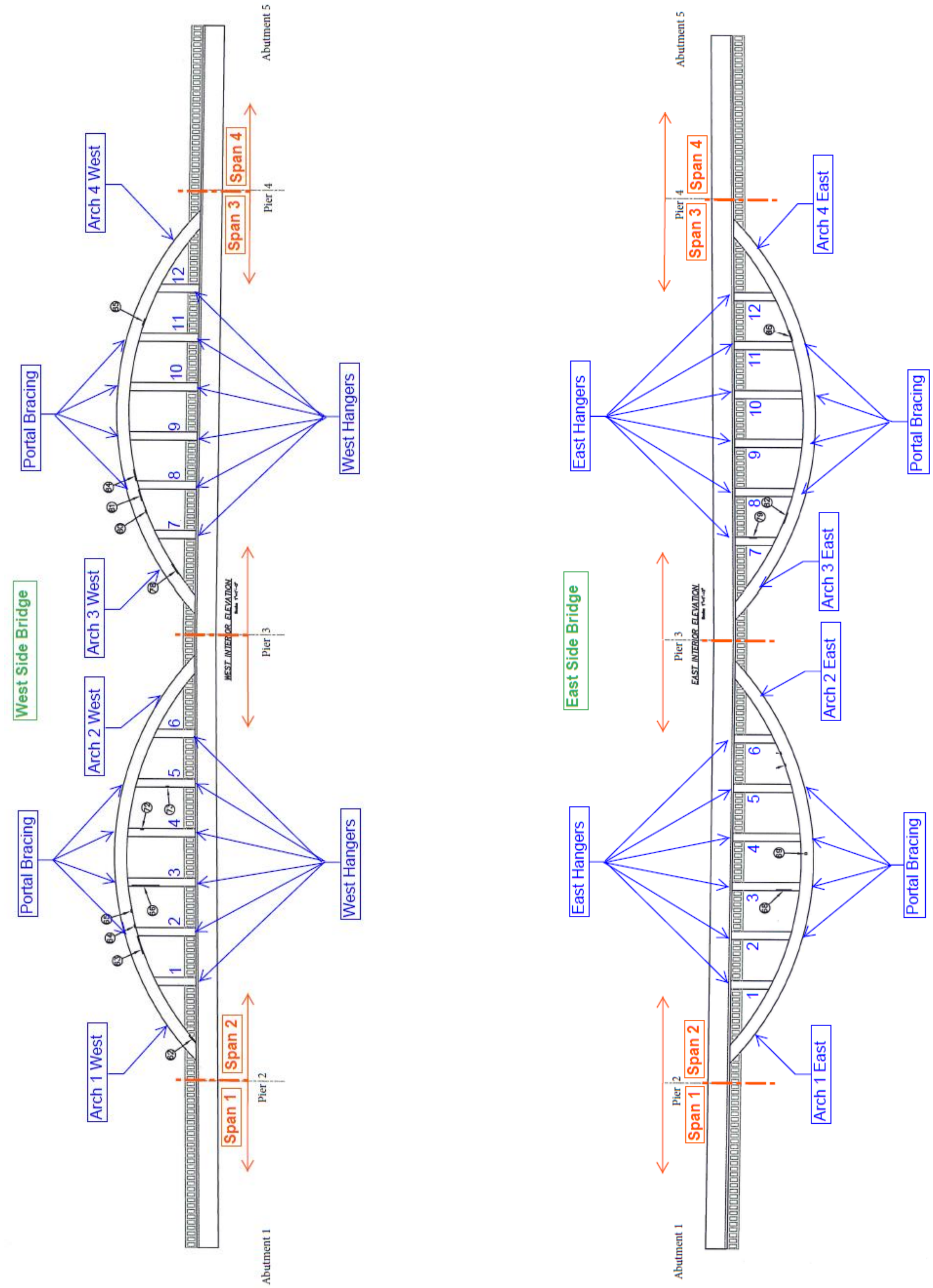


Figure 3 - *Section of bridge identifying different members and elements on west side of bridge*

B. Visual Inspection

Unmanned Aircraft System Imaging

Rather than close the bridge and use man lifts and personnel to physically climb on the bridge to perform the technical assessment, Alta Vista utilized an unmanned aircraft system (UAS) carrying a remote sensor to collect a comprehensive image data set of the bridge. Overall about 6,000 images were collected during UAS imaging operations. Images were processed to generate a suite of seven (7) ortho mosaic images, a 3D mesh model, and a 3D point cloud model that were used by the technical engineering team to analyze and assess soffit, transverse floor beams, hanger columns, arches and portals of the bridge. Figures 4 through 6 are examples of images produced from this inspection.

Figure 4 - UAS rendering of Arch 1 East

Figure 5 - UAS rendering of Arch 3 West

Figure 6 - Underside view at Span 1 rendered from several images

Visual Observations and findings

Visual inspections, review of images from UAS operations and other digital photography, and input from Solano County and Quincy Engineering Inc. were utilized to select areas of the bridge to perform further inspections. Based on observations such as large transverse cracks or exposed reinforcement on vertical hangers/arches; operations for borescope, concrete coring, and GPR were planned accordingly.

Below is a summary of initial observations made.

- Both approach spans (Spans 1 and 4) show significant structural defects including:
    - Major transverse cracks in the deck at each approach span extend down into the supporting girders as major vertical cracks in the girders. The cracks occur 3/4 of the way into the span towards the piers (away from the abutments). TRC Imbsen notes these flexural cracks in their Feasibility Study, as well as in a "Field Review Report" that was previously submitted to the County. The cracks are cited alongside a number of other defects which may have a direct effect on the service life of the structure.
    - Based on the existing reports, the cracks could be more than 10 years old and appear to have resulted from settlement at the abutments and/or loads from the arch span causing negative moments on the approach span. See Figures 7 and 8 for images of the crack locations, which are identified by blue paint on either side. Aerial views of these areas can be found in **Appendix 4**.
- Spans 2 and 3 also show major signs of distress including:
    - Major spalls with exposed rebar in numerous bays and transverse floor beams.
- Several vertical hanger columns have significant defects
- The arches have significant defects at several locations
- The columns and piers don't show significant signs of defects on the exterior
- The abutments appear to have settled and cracked in the corners
- It appears there is no top mat reinforcement in girders in Spans 1 and 4.
- It is unclear if the shear reinforcement is full length for the girders in Spans 1 and 4.

Figure 7 - Location of crack between Abutment 1 and Pier 2 as shown through UAS image

Figure 8 - Location of crack between Pier 4 and Abutment 5 as shown through UAS image

Repair Recommendations for Approach Spans

Assuming no further settlement is anticipated at the approach spans, and no enhanced member capacity is needed, the following repair strategy may be employed. Cracks in the approach spans are considered full depth repairs and should be repaired by removing deteriorated concrete up to six feet north and six feet south of the crack locations, and reconstructing the bridge deck.

For bridge deck, if removal of deteriorated material requires saw-cutting, existing reinforcement should not be damaged. This may be achieved by chipping or hydro blasting, which should employ appropriate equipment that will not damage surrounding concrete or steel. Demolition should result in repair areas that have a step configuration to allow mechanical engagement. Added reinforcement may be required where reinforcement condition appears to be damaged due to settlement, corrosion, or other causes. The Engineer should witness removal operations in order to verify that the extents of damage have been removed, or if further removal is needed. Prior to repairs, surfaces should be cleaned of all substances that would impair bond of repair materials, and an SSD surface condition may be required prior to placement of repair material.

For repair of the girders affected by this cracking, it is recommended that loose material be removed, which may extend 1 inch below the first layer of reinforcement. Areas where cracking is present should be opened to expose sound material. As with the deck, care should be taken to avoid damage to the steel. If the condition of concrete and steel appear deteriorated and extends deeper into girder than is shown from the surface, notify the Engineer to assess the condition and determine an appropriate repair method with additional reinforcement.

Estimated Deck Repair Area:	775 sq.ft.
Girder Estimated Repair Area:	40 sq.ft.
Estimated Reinforcement:	100 ft

Repair Recommendations for Soffit

Visual inspections and documentation of defects were performed for all four spans of the bridge. A numbered listing of all documented defects by span number are available in **Appendix 2** along with diagrams showing the location of the defect. Images of all defects are catalogued in this section to provide a visual guide. Repairs are recommended based on the severity of the defect noted. Table 2 provides recommendations for addressing each types of defects identified.

Table 1 - Repair methods for soffit defects

Category 1 **GOOD**	Generally, no defects identified. No repair required, however it is recommended that visual inspection be performed after any substructure retrofit is complete or as deck repairs are being done to assess whether any additional defects result. At this point, reassessment of defect category must be performed and applicable repairs be performed as needed.
Category 2 **FAIR**	At the locations identified with cracking or rocks pockets/voids, repairs should include removal of unsound concrete, saw-cutting two inches beyond the affected area. Saw-cut for overhead repairs shall be angled to promote mechanical engagement with of repair material with existing. If, during removal of unsound concrete, reinforcement is exposed, follow the repair procedure for Category 3/4. If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010” should be repaired by epoxy injection or other suitable material. Proper surface preparation and bonding agent should be employed based on manufacturer’s recommendations for appropriate patching material.
Category 3 **POOR**	At the locations identified with cracking, exposed reinforcement, or rock pockets/voids, repairs should include removal of unsound concrete, and saw-cutting two inches around the affected area. Saw-cut for overhead repairs shall be angled to promote mechanical engagement with of repair material with existing. In case of exposed rebar, material removal should extend 1 inch beyond the first layer of reinforcement to allow mechanical engagement of repair material.
Category 4 **SEVERE**	After material removal is complete, exposed reinforcement should be cleaned of bond inhibiting agents and concrete should be examined for cracks. If, during removal of concrete it is determined that cross-section loss has occurred, notify the Engineer to determine appropriate repair method. If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010” should be repaired by epoxy injection or other suitable material. Proper surface preparation and bonding agent should be employed based on manufacturer’s recommendations for appropriate patching material.

Estimated Soffit Repair Area:	Category 2:	267 sq.ft.
	Category 3:	380 sq.ft.
	Category 4:	759 sq.ft.

Repair Recommendations for Arches, hangers and railings

Various observations of defects were recorded for the superstructure of the bridge. Individual locations from the superstructure are shown in Table 3.

In general, locations which have exposed reinforcement and spalled or loose material as shown in Figures 9 and 10 need to be repaired, which include removing loose material until sound concrete is encountered, cleaning rebar and concrete substrates, and applying patching material to restore the surface of the member while protecting the rebar from corrosion.

Figure 9 - Exposed rebar at east Abutment 1 railing

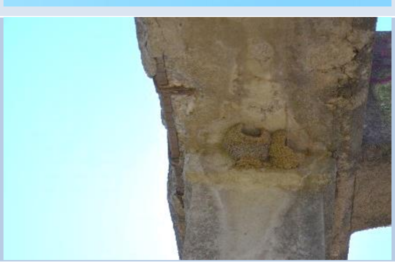
Figure 10 - Spalled overhead section of arch

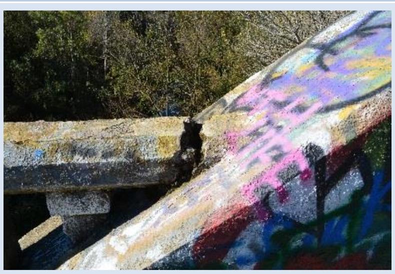
If, during removal of unsound concrete cracks are observed, those cracks should be measured. Cracks larger 0.010" should be repaired by epoxy injection. Typically, available epoxy products have a range of viscosities available which are able to accommodate repairs to cracks of up to 1/4 inch width.

Table 3 identifies deteriorated areas observed on the superstructure and potential repair strategies that may be used. The Feasibility Study provided recommendations for retrofit including fiber wrap for seismic loading. While fiber wrap is commonly used to increase strength and confinement, the repairs recommended here including patching and fiber wrap are intended to protect the identified element from further deterioration and to restore to as-built conditions.

Estimated Repair Area: 37 sq.ft. plus lumpsum for railing

Table 3 - Summary of defects on superstructure and potential repair strategies.

Element	Image	Location and Description
Hangers		Hanger 5 West (Repair area: 6 sq.ft.) Exposed reinforcement, heavy spalling, and visible aggregate. *Remove unsound material and bond inhibiting substances. Clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*
		Hanger 11 East (Repair area: 6 sq.ft.) Heavy spalling, cracking, unsound concrete. *Remove all unsound material, clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*
Arches		West at Hanger 7 (Repair area: 4 sq.ft.) Exposed reinforcement under arch, cracking and spalling *Remove unsound concrete, clean rebar and patch. Fiber wrap at this location due to proximity to pier.*
		West at Hanger 8 (Repair area: 6 sq.ft.) Exposed reinforcement under arch, cracking, heavy spalling, loose material. *Remove all unsound material, clean rebar and patch. Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*
		West at Hanger 11 (Repair area: 4 sq.ft.) Exposed reinforcement, cracking, some spalling, possible loose material *Remove unsound concrete, clean rebar and patch.*
		East at Hanger 11 (Repair area: 6 sq.ft.) Exposed reinforcement, cracking, spalling, loose material. *Fiber wrap at this location if loss of cross section for reinforcement is discovered, or if seismic calculations require additional capacity.*

Element	Image	Location and Description
Portal Bracing		Bracing at Hanger 2 (Repair area: 2 sq.ft.) Exposed reinforcement and spalling *Remove unsound concrete, clean rebar and patch. Use fiber wrap or wire reinforcement to secure overhead patch material from falling.*
		Portal Bracing at Hanger 8: (Repair area: 3 sq.ft.) Corner spalling, cracking, exposed reinforcement. *Remove unsound concrete, clean rebar and patch. Use fiber wrap or wire reinforcement to secure overhead patch material from falling.*
Railing		Between Abutment 1/Pier 2 (Repair area: lumpsum) Spalled railing posts, exposed reinforcement. *Remove unsound material, clean surface and patch. If majority of section is damaged, individual posts should be replaced in kind.*
		At Arch 1 West (Repair area: lumpsum) Cracking at what appears to be patched area. *Remove unsound material, clean surface and patch.*
		At Arch 3 west (Repair area: lumpsum) Crack and void between railing and arch. *Remove unsound material, clean surface and patch.*
		At Arch 4 East (Repair area: lumpsum) Appears to be an uneven repair area. *Remove unsound or uneven material, clean surface and patch.*

C. Borescope Observation

The intent of utilizing borescope on the bridge was to collect observations from small holes that are drilled above reinforcement and other locations. This inspection method uses a small optical tip attached to a probe and tube which is connected to handheld device with a LCD screen for real-time observations during inspection. The equipment used for this investigation was a GE XLGO 6120 (see Figures 11 and 12). The borescope was used to inspect for signs of corrosion or cracking at 9 locations on the bridge. All borescope locations are shown on the annotated work location diagram included in **Appendix 1**. Table 4 provides a summary of observations taken during borescope inspection. Images from each location can be found in Figures 13 through 15 below and in **Appendix 3**.

Figure 11 - Borescope equipment used for field inspection

Figure 12 - Use of borescope at Pier 3 west side

Table 4 - Summary of borescope observations

Location	Description	Depth	Notes
B1	Bridge Deck at Pier 2	2 in	Placed above rebar. No apparent cracking or rusting noted.
B2	Bridge Deck at Pier 2	10 in	No apparent cracking noted.
B4	Bridge Deck at Pier 3	8 in	Drilled through full deck thickness. No apparent cracking noted.
B5	Bridge Deck at Pier 4	7 in	No apparent cracking noted.
B6	Bridge Deck at Pier 4	7 in	Slight color variation at about half depth, possibly due to coarse aggregate color.
B7	Pier 3 Under Bridge	10 in	No apparent cracking noted.
B8	Hanger 10 West	9 in	Placed above rebar. No apparent cracking or rusting noted.
B9	Arch 3 East	10 in	Possible material consistency difference 1-2 inches down, possibly due to drill pattern.
B10	Hanger 5 West	2 in	Placed above rebar No apparent cracking or rusting noted.

Based on the limited observations from the borescope locations surveyed, no defects such as concrete cracking or reinforcement corrosion were identified in order to make recommendations for repairs to the respective elements.

Figure 13 - Hanger 5 West borescope location

Figure 14 - Hanger 10 West borescope location

Figure 15 - Arch 3 East borescope location

D. Ground Penetrating Radar (GPR)

Overview

The ground penetrating radar survey method is a non-destructive inspection method that utilizes equipment which sends electromagnetic radar pulses into a surface and records the reflected waveforms. As the radar pulses through a material, the reflected waves bend slightly as they encounter materials with different physical properties. These properties include conductivity (dielectric) and density. In this investigation, the varying properties that may be encountered in existing concrete may include rebar, conduit, air, or other indications.

The equipment used for performing GPR scanning of the bridge deck, arches, vertical hangers, and Pier 4 column was a GSSI SIR 3000 with an antenna frequency of 1,600 MHz. GPR was utilized on the bridge to locate rebar and assist with obtaining samples for compressive strength, along with laying out locations for borescope holes. For data collection, the GPR equipment was used to scan various areas to identify indications or inconsistencies within the various elements.

Bridge Deck (Large) Areas

Table 5 summarizes the bridge deck locations at Span 1 and Span 4. Individual scans were reviewed and summarized in Tables 6 and 7 for Span 1, and Tables 8 and 9 for Span 4. Figures 20 and 22 are diagrams of the scan lines, which show individually numbered lines that correspond with the tabulated line numbers. Additional aerial views and individual scan images can be found in **Appendix 4 & 5.**

Table 5 - Bridge deck GPR locations

GPR Location	Notes
Bridge Deck near Abutment 1 / Pier 2 (Figure 16)	• Scan area 18' W x 12' L with 1' scan spacing. South to north scans. • Transverse scans at selected locations. West to east scans. • Scan area encompasses a large transverse crack near Pier 2.
Bridge Deck near Pier 4 / Abutment 5 (Figure 17)	• Scan area 18' W x 13' L with 1' scan spacing. South to north scans. • Transverse scans at selected locations. West to east scans. • Scan area encompasses a large transverse crack near Pier 4.

Figure 16 – GPR scan area at Abutment 1/Pier 2

Figure 17 – GPR scan area at Pier 4/Abutment 5

General Observations for Bridge Deck Abutment Locations

- In areas that encounter reinforcement, objects beneath top layer reinforcement may be obscured in data output due to absorbed signal reflections from shallower objects.
- The deck appears to be about 8 inches thick; the as-builts do not provide this information.
- Concrete generally appears to have consistent appearance with the exception of anomalies

identified at locations in Tables 6 through 9. Anomalies are parabolic GPR indications that appear out of place in a scan. They can be conduit, rebar, or other items embedded in concrete and are identified when they appear outside of an expected pattern (i.e. a rebar mat).

- Scan lines at the east- or west-most edges of the deck appear to have a bottom layer of reinforcement near the bottom of the slab. As scans move towards the road centerline, bottom layer rebar placement is shown higher towards the center of the deck. The lower transverse rebar appears to span the width of the bridge as shown in the as-built plans.
- Top layer transverse rebar does not appear to span the entire width, and appears to exist at Lines 3 to 5, 9 to 11, and 15 to 17 for both locations. This is consistent with the location of longitudinal rebar at similar depth shown in other scans.
- Example scans are shown in Figure 18 and 19. The horizontal scale is in feet and the vertical scale is inches below the surface. Figure 18 is at Span 1 (Line 7), and shows a small anomaly about 5 feet along the scan, and about 5 inches deep. Another small anomaly can be seen at about the same depth, at 10 feet along the scan. The blue lines on each figure represent material separation, such as the bottom of the deck in Figure 18 and the possible interface between the deck and girder in Figure 19. Orange dots show locations of rebar in Figure 17. For clarity, rebar was not marked in Figure 18, as it is shown close to the bottom of the deck.

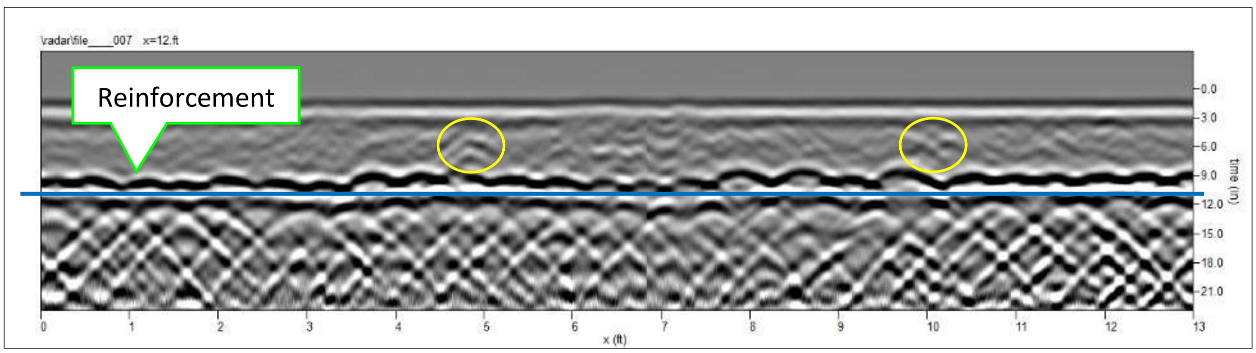


Figure 18 - Example GPR scan from Abutment 1 (Line 7). Shows small anomalies at about 5 and 10 feet, both 5 inches deep

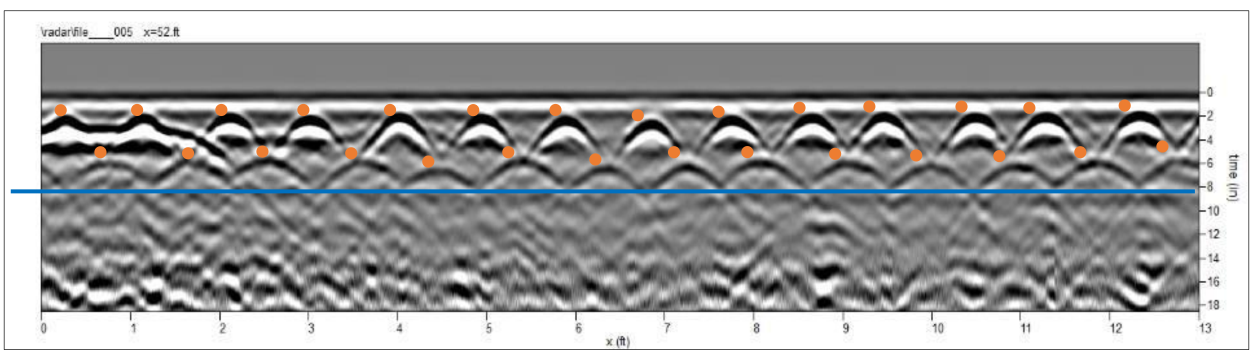


Figure 19 - Example GPR scan from Abutment 5 (Line 5). Shows rebar locations and possible location of girder

<u>Span 1 – Abutment 1 to Pier 2</u>

The following aerial view and tables summarize the observations made at Span 1. As a reference, the location of the transverse crack exposed from the top surface is about 5 to 7 feet from the southern edge of the scan area as highlighted in Figure 18. A second major crack is on the east side of the scan area about 8 to 9 feet from the southern edge of scan area.

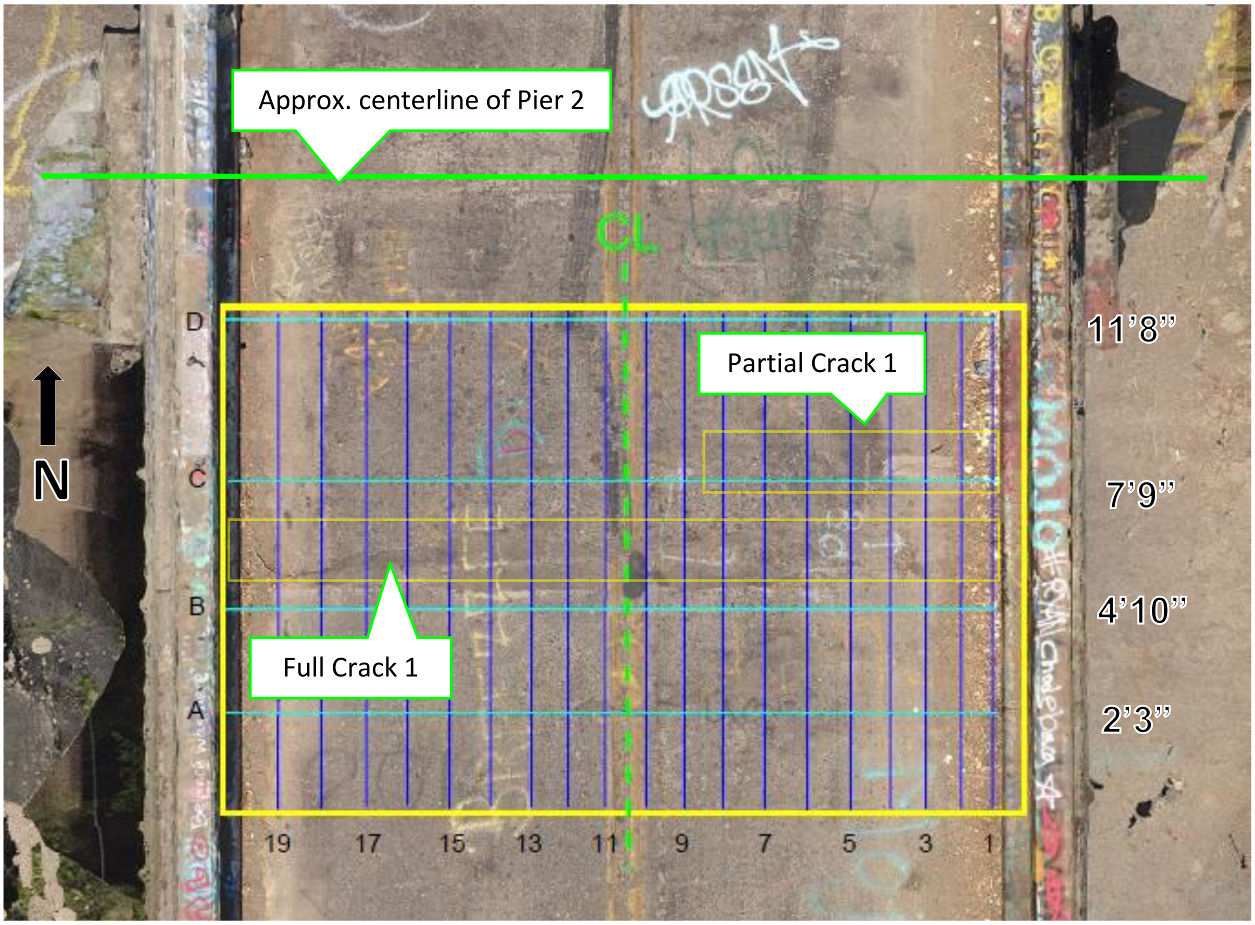


Figure 20 - Approximate GPR scan lines at Abutment 1/Pier 2 bridge deck

For scans in the longitudinal direction at Span 1 bridge deck, there are some locations where anomalies appear to coincide with the transverse cracks or with a defect identified during visual inspection of the soffit. These are indicated in Table 6 by underlined text and a defect number, if applicable, showing where adjacent scans have indications at similar location and depth, in this case Lines 6-8, Lines 12-14, and Lines 18-19. At these locations, an indication under the surface spanned across two or more scans. One location at the north edge of the scan area showed adjacent indications across three scans. While this location did not match with a documented soffit defect, it could be additional reinforcement considering its proximity to Pier 2.

All images of GPR scan lines can be found in **Appendix 5**.

Since limited scans were documented in each direction, small anomalies that appear at a single location (not across adjacent scans) may be considered innocuous. Without complete data in the transverse direction, it is not possible to determine the extents of the anomaly, unless it is also identified during soffit inspection.

Table 6 - Bridge Deck at Abutment 1/Pier 2 GPR observations – Longitudinal Scans

Location ID	Observations	Area of Interest
Line 1	• No transverse reinforcement encountered in top layer	
Line 2	• No transverse reinforcement encountered in top layer • X=1 ft: Small anomaly at about 3" deep	
Line 3	• Transverse reinforcement encountered about 2-3" • Second reinforcement layer approximately 6-7" deep	
Line 4	• Transverse reinforcement encountered about 2-3" deep • Second reinforcement layer approximately 6-7" deep • Possible layer of longitudinal reinforcement about 2-3" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 5	• Transverse reinforcement pattern similar to Line 4 • X=10 ft: Possible layer of longitudinal rebar appears thru end of scan • Scan location may be on top of girder based on GPR signal pattern	
Line 6	• No transverse reinforcement encountered in top portion • <u>X=8.5 ft: Small anomaly at about 6" deep. Coincides w/ Partial Crack 1.</u> • <u>X=10 ft: Small anomaly at about 4" deep. Coincides w/ Defect 113.</u>	**X**
Line 7	• No transverse reinforcement encountered in top layer • <u>X=5 ft: Small anomaly at about 5" deep. Coincides w/ Full Crack 1.</u> • <u>X=10 ft: Small anomaly at about 5" deep. Coincides w/ Defect 113.</u>	**X**
Line 8	• No transverse reinforcement encountered in top layer • <u>X=5 ft: Small anomaly at about 5" deep. Coincides w/ Full Crack 1.</u>	**X**
Line 9	• Layer of transverse reinforcement encountered about 4-5" deep • Second reinforcement layer approximately 6-7" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 10	• Transverse reinforcement pattern similar to Line 9 • X=9 ft: Possible layer of longitudinal rebar appears through end of scan • Scan location may be on top of girder based on GPR signal pattern	
Line 11	• Transverse reinforcement pattern similar to Line 9 • Possible layer of longitudinal rebar at about 4-5" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 12	• No transverse reinforcement encountered in top layer • <u>X=12.5 ft: Medium anomaly at about 6" deep, may be single rebar</u>	**X**
Line 13	• No transverse reinforcement encountered in top layer • <u>X=7 ft: Small anomalies near surface. Coincides w/ Full Crack 1.</u> • <u>X=12 ft: Small anomaly about 6" deep</u>	**X**
Line 14	• No transverse reinforcement encountered in top layer • <u>X=5.5 ft: Small anomaly about 6" deep. Coincides w/ Full Crack 1.</u> • <u>X=7 ft: Small anomaly at about 5" deep. Coincides w/ Full Crack 1.</u> • <u>X=12.5 ft: Small anomaly at about 6" deep</u>	**X**

Location ID	Observations	Area of Interest
Line 15	• Layer of transverse reinforcement encountered about 3-4” deep • Second reinforcement layer approximately 8” deep • Possible layer of longitudinal rebar at about 3-4” deep. Shown at beginning of scan up to 6 feet.	
Line 16	• Transverse reinforcement pattern similar to Line 15 • Scan location may be on top of girder based on GPR signal pattern	
Line 17	• Transverse reinforcement pattern similar to Line 15 • Scan location may be on top of girder based on GPR signal pattern	
Line 18	• No transverse reinforcement encountered in top portion • <u>X=6 ft: Small anomaly at about 6” deep. Coincides w/ Full Crack 1.</u> • X=8 ft: Medium anomaly at about 5” deep, may be single bar of rebar	**X**
Line 19	• No transverse reinforcement encountered in top portion • X=3 ft: Small anomaly at about 3” deep, Coincides w/ Defect 105. • <u>X=6 ft: Small anomaly at about 6” deep. Coincides w/ Full Crack 1.</u>	**X**

Observations for Span 1 transverse scans are shown in Table 8, with the location identified as the distance from the southern edge of the scan area. For scans in the transverse direction, there was one indication at Line B that appears to be an air pocket or other hollow embedment (Figure 21). This area coincides with defect #114 which was identified during soffit visual inspection. A limited number of scans were performed in the transverse direction, so the quantity and extent of indications cannot be defined as is done in the longitudinal direction. All images of GPR scan lines can be found in **Appendix 5**.

Table 7 - Bridge Deck at Abutment 1/Pier 2 GPR observations – Transverse Scans

Location ID	Observations	Area of Interest
Line A (at 2'3")	● Staggered longitudinal rebar patterns centered at 4 ft, 10 ft, and 16 ft along the scan. See Figure 19. ● X=1.5 ft: Small anomaly at about 3" deep	
Line B (at 4'10")	● Longitudinal reinforcement pattern similar to Line A ● X=0.5 ft: Small anomaly at about 4" deep ● X=13 ft: Medium anomaly at about 4" deep. Coincides with Defect 114	X
Line C (at 7'9")	● Longitudinal reinforcement pattern similar to Line A ● X=5.5 ft: Small anomaly at about 5" deep. Coincides w/ Partial Crack 1. ● X=8 ft: Small anomaly at about 5" deep. Coincides w/ Partial Crack 1.	
Line D (at 11'8")	● Longitudinal reinforcement pattern similar to Line A, with additional rebar in top layer ● X=1.5 ft: Small anomaly at about 3" deep	

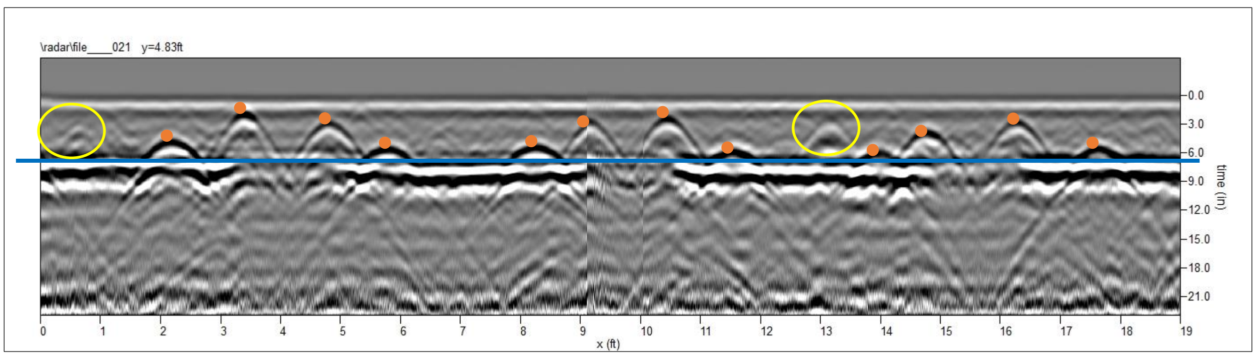


Figure 21 - Transverse scan at Abutment 1, Line B. Anomalies are circled.

Span 4 – Pier 4 to Abutment 5

The following aerial view and tables summarize the observations made at Span 4. As a reference, the location of the transverse crack seen from the top surface is about 7 to 8 feet from the southern edge of the scan area.

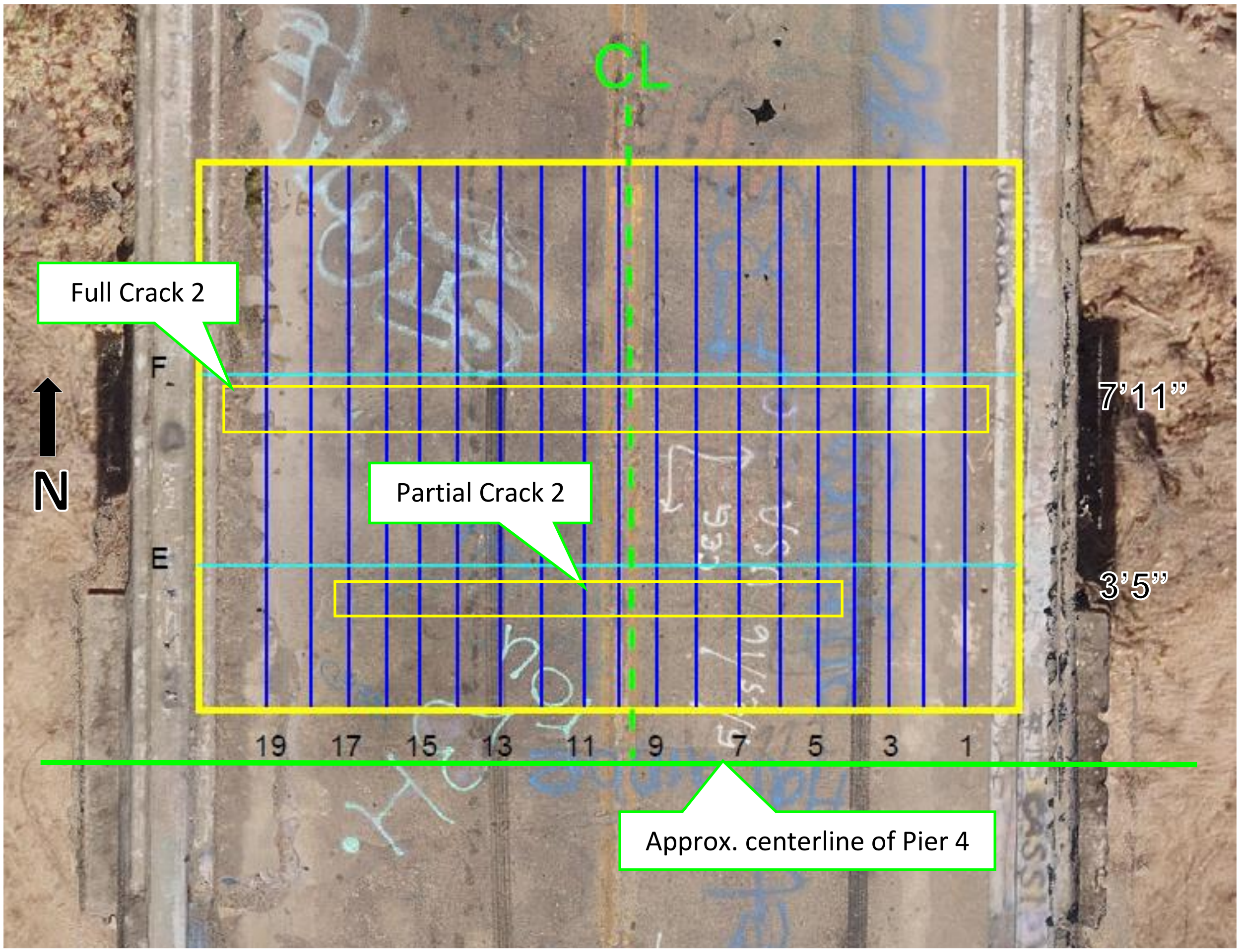


Figure 22 - Approximate GPR scan lines at Pier 4/Abutment 5 bridge deck

Similar to Span 1, Table 8 shows similar observations for Span 4. There were a few locations at Pier 4/Abutment 5 that exhibited indications spanning across more than one scan line. Potential areas for further investigation or repair include those at Lines 1-2, and Lines 8-12. The indications at these locations appear relatively shallow (within 3" from the surface), and if concluded to be a defect, may consider repair by removal of unsound material and spall or deck treatment.

Table 8 – Bridge Deck at Pier 4 / Abutment 5 GPR observations – Longitudinal Scans

Location ID	Observations	Area of Interest
Line 1	• No transverse reinforcement encountered in top layer • X=9.5 ft: Small anomaly at about 3" deep. Coincides w/ Defect 139, 140.	X
Line 2	• No transverse reinforcement encountered in top layer • X=10 ft and 11 ft: Small anomalies at about 3" deep. Coincides w/ Defect 139, 140.	X
Line 3	• Layer of transverse reinforcement encountered about 3-4" deep • Second reinforcement layer approximately 6" deep	
Line 4	• Layer of transverse reinforcement encountered about 2-3" deep • Second reinforcement layer approximately 6" deep • Scan appears to run along a longitudinal rebar about 3" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 5	• Layer of transverse reinforcement encountered about 2-3" deep • Second reinforcement layer approximately 6" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 6	• Transverse reinforcement appears to show in some locations 3" deep and does not show in others; may be along the edge of rebar ends • X=11 ft: Small anomaly at about 2" deep	
Line 7	• No transverse reinforcement encountered in top layer • X=2 ft: Medium anomaly at about 4" deep, may be single rebar end	
Line 8	• No transverse reinforcement encountered in top layer • X=8.5 ft: Small anomaly at about 3" deep – close to Full Crack 2 • X=10.5 ft, 11 ft: Small anomalies about 3" deep, Coincides w/ defect 149, 150.	X
Line 9	• Layer of transverse reinforcement encountered about 3-4" • Second reinforcement layer approximately 6" deep • X=9.5 ft, 10.5 ft: Small anomalies at about 3" deep, Coincides w/ defect 149, 150.	X
Line 10	• Transverse reinforcement pattern similar to line 9 • X=10.5 ft: Small anomaly at about 3" deep, Coincides w/ defect 149, 150. • Scan location may be on top of girder based on GPR signal pattern	X
Line 11	• Transverse reinforcement pattern similar to line 9 • Scan location may be on top of girder based on GPR signal pattern	
Line 12	• No transverse reinforcement encountered in top layer • X=11 ft: Small anomaly at about 3" deep, Coincides w/ defect 149, 150.	X
Line 13	• No transverse reinforcement encountered in top layer • X=3.5 ft: Small anomalies about 3" deep	

Location ID	Observations	Area of Interest
Line 14	• No transverse reinforcement encountered in top layer • X=8 ft: Small anomaly at about 4" deep • X=12 ft: Medium anomaly at about 4" deep. Likely single rebar	
Line 15	• Layer of transverse reinforcement encountered about 2-3" deep • Second reinforcement layer approximately 6" deep • Scan appears to run along a longitudinal rebar about 3" deep • Scan location may be on top of girder based on GPR signal pattern	
Line 16	• Transverse reinforcement pattern similar to Line 15 • Scan location may be on top of girder based on GPR signal pattern	
Line 17	• Transverse reinforcement pattern similar to Line 15	
Line 18	• No transverse reinforcement encountered in top layer, except at X=2.5 ft which may be a single rebar • X=10 ft: Small anomaly at about 4" deep	
Line 19	• No transverse reinforcement encountered in top portion	

Observations for Span 4 transverse scans are shown in Table 9, with the location identified as the distance from the southern edge of the scan area.

Table 9 - Bridge Deck at Pier 4 / Abutment 5 GPR observations – Transverse Scans

Location ID	Observations
Line E (at 3'5")	• Staggered longitudinal rebar patterns centered at 3 ft, 9 ft, and 13.5 ft along the scan (Figure 23).
Line F (at 7'11")	• Longitudinal reinforcement pattern similar to Line E (Figure 23).

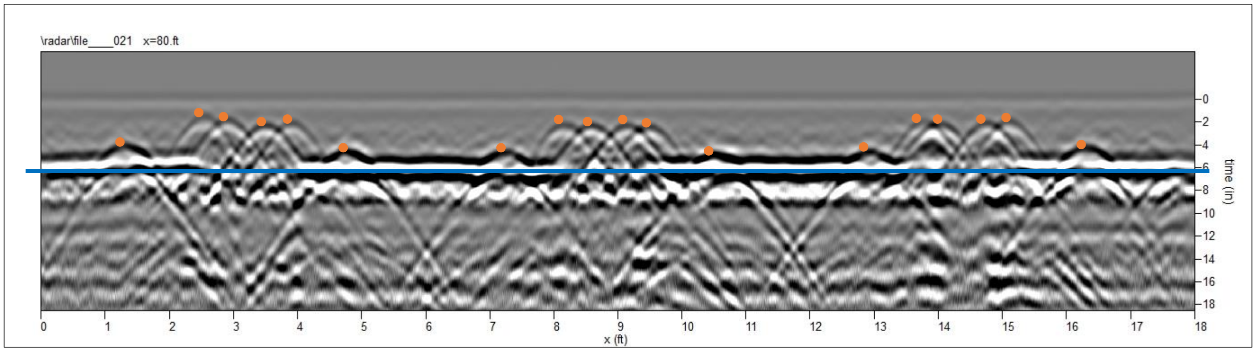


Figure 23 - Transverse scan at Span 4, Line E

GPR observations and findings

Based on review of the data across longitudinal transverse scans at Span 1 and 4, a majority of anomalies detected coincided with the locations of major transverse cracks. If the approach span repair strategy is implemented, all of these indications would be addressed under those repairs.

If less than six feet on either side of the full-length cracks 1 or 2 is decided to be replaced, some additional considerations can be taken. For locations with indications across two or more scans which are not near a crack, some coincided with the locations of defects identified during soffit inspection. This information should be verified during repairs on the soffit areas, to ensure that the indications captured by GPR are also repaired, if necessary. For the remaining single locations showing an anomaly, there is not enough information to determine if it is a defect if it does not coincide with a visual observation from the soffit. There is not enough information to determine if additional repairs are needed in these locations.

Small Areas (Arches, Columns, Pier)

In addition to the large scan areas, five small scan locations were selected to include the arches, vertical hangers, and one of the Piers. All small area scans were taken on the face of each element adjacent to the roadway. For the hanger columns, the scan area was limited on one side due to space requirement for the GPR equipment to grip rolling surface. In order to obtain post-processed images, a standard 24” W x 24” H template was utilized for all scan areas. Table 10 provides a summary of observations taken from GPR data post-processing.

Table 10 - Observations at small GPR scan areas

GPR Location	Observations
Overall observations	• Location of reinforcement indicated by pink lines in each figure. • Vertical and horizontal scales are in feet (up to 2 ft shown) • Slightly differing physical properties indicated by dark shading. This may result from variation of concrete consolidation based on distance from reinforcement (i.e. more paste concentration closer to reinforcement, and more coarse aggregate concentration in areas further)
Hanger 11 East (NB side) *See Fig 24 and 26*	• Selected based on spalling on column • 12” W x 24” H with 2” scan spacing • First rebar layer ~4” deep. Second rebar layer ~8” deep. • Possible plate located in top scan area (Fig. 24). • Possible material variation at 8” - 9” deep which coincides with depth of lower reinforcement layer. 

GPR Location	Observations
Hanger 5 West (SB side) *See Fig 25 and 27*	● Location selected based on heavy spalling on column ● 12" W x 24" H with 2" scan spacing ● First rebar layer ~3" deep. Second rebar layer ~8" deep. ● Possible, hard to see material variation at ~8" deep which coincides with depth of lower reinforcement layer. 
Arch 2 East at Spring Line (NB) *See Fig 28 and 30*	● 24"x24" with 2" scan spacing ● First rebar layer ~2" deep. Second rebar layer ~4" deep. ● Possible material variation at ~8.2" deep with another material variation layer at ~14.3" deep. 
Arch 3 West at Spring Line (SB) *See Fig 29 and 31*	● Scan area adjacent to concrete core location ● 24"x24" with 2" scan spacing ● First rebar layer ~2.5" deep. Second rebar layer ~8.2" deep. ● Possible, faint material variation at ~9.2" deep which may coincide with lower reinforcement layer. 
Pier 4 (NB side) *See Fig 32*	● 24"x24" with 2" scan spacing ● First rebar layer ~2" deep for both horizontal and vertical directions. ● Possible material variation at ~8.2" deep. 

Based on review of the processed data for each of the locations, it appears there are no major defects observed in the areas inspected. While slight material variation are identified at all locations, this may be due primarily to slight differences in physical properties in the material. For example, at Arch 3 West a vague material variation was detected about 9.2" deep. At this same location a concrete core over 10" long was extracted that did not show any obvious interface at that depth. Based on GPR observations noted for the areas above, no additional repairs are recommended.

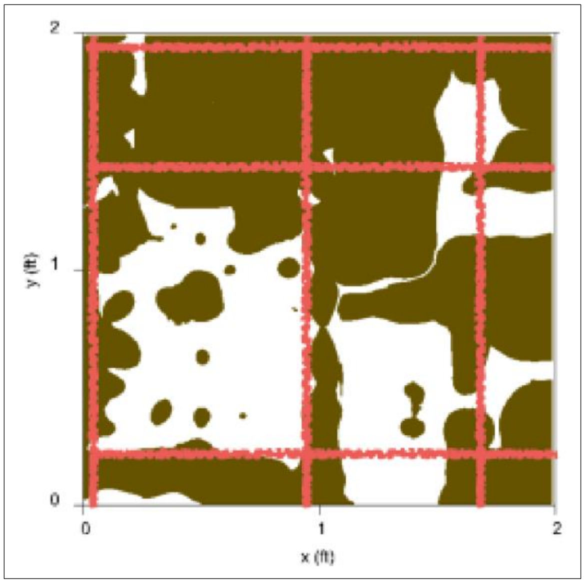


Figure 24 – Hanger 11 East

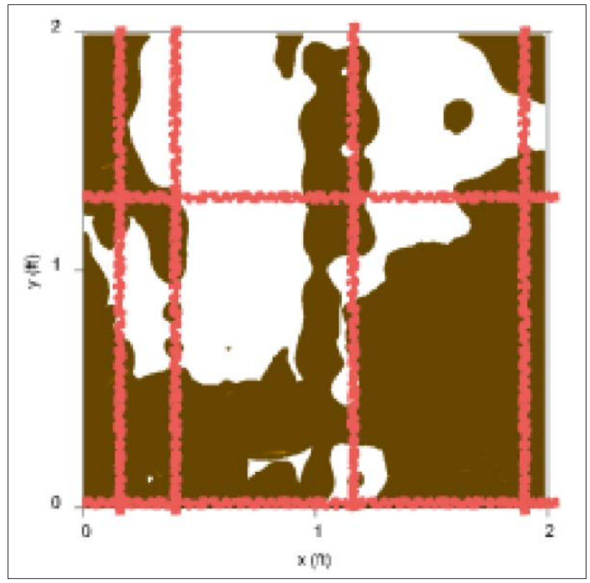


Figure 25 – Hanger 5 West

Figure 26 – Hanger 11 East

Figure 27 – Hanger 5 West

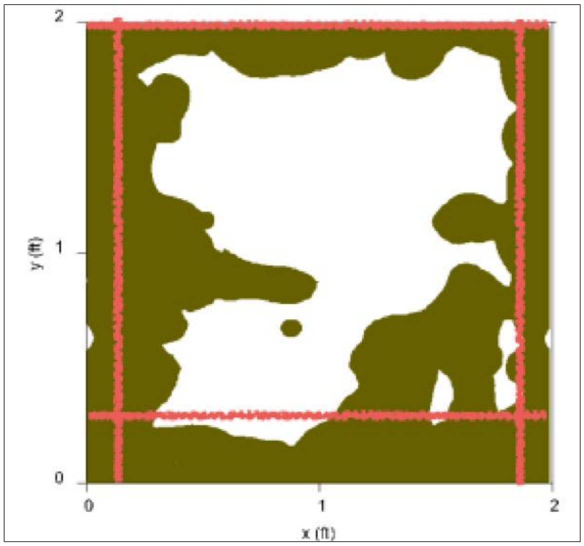


Figure 28 – Arch 2 East

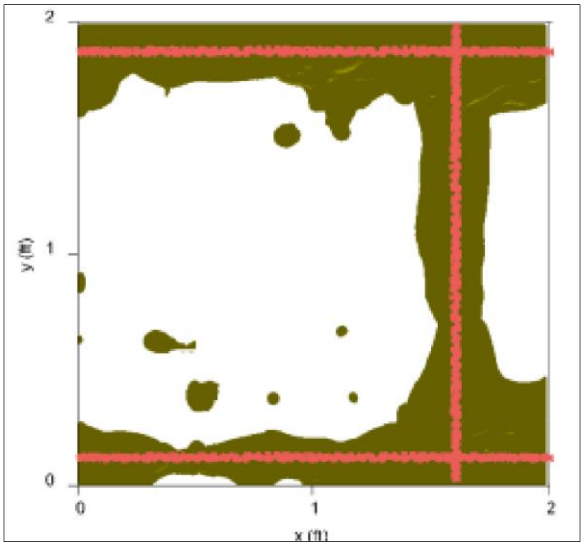


Figure 29 – Arch 3 West

Figure 30 – Arch 2 East

Figure 31 –Arch 3 West

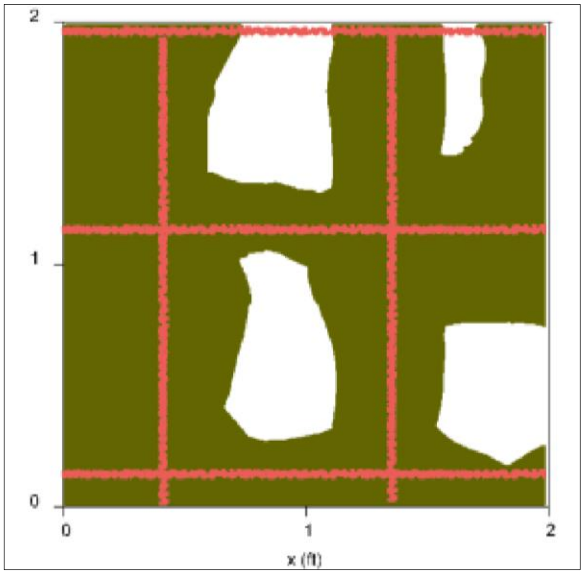


Figure 32 – Pier 4

E. Concrete Core Testing

Concrete samples for compressive strength were extracted from four locations on the bridge, including the arches, bridge deck, and from one of the girders. Cores were sampled and cured per ASTM C-42 and were tested in accordance with ASTM C-39. Table 11 provides the length, diameter, and resulting compressive strength for each of the cores extracted. During coring, no signs of delamination were noted and no reinforcement was encountered. A copy of the compressive strength data is attached in **Appendix 6**.

After the samples were taken and secured, the core holes were repaired using BASF Set 45 per manufacturer's recommendations.

Table 11 - Concrete compressive strength testing by Alta Vista

Sample Number	Length (in.)	Diameter (in.)	Description	Compressive Strength (psi)
1B	4.70	3.63	Arch 1 East near the spring line	3,560
2	7.48	3.63	Arch 3 West near the spring line	2,860
3	5.12	2.66	Bridge Deck at Pier 4/Abutment 5 near major crack location	3,490
4	5.02	2.66	West girder at Abutment 1	3,670

Where possible, locations tested were compared with the compressive strengths reported in Kleinfelder's test reports dated January 25, 2006 (Tables 12 and 13). Sample 3 was taken from a similar bridge deck area as Kleinfelder's sample S4-B, and reported strengths are within 10% of each other.

Kleinfelder also performed Schmidt Hammer testing per ASTM C805 at various locations on the structure, including the arches, retaining wall, abutment, and bents. In terms of reliability of a nondestructive method such as the Schmidt Hammer, while it may be a fairly reliable means of estimating compressive strength and general concrete condition, testing should always be supplemented by compressive strength testing per ASTM C42. Schmidt Hammer testing results are typically influenced by factors that affect surface hardness, environmental exposure, and proper calibration. These factors have shown to result in a wide dispersion of data. Therefore, this report does not take into account those data.

Based on testing performed on the specimens collected, it can be established that for the sound concrete the bridge deck and girders have an average compressive strength of about 3,500 psi. The arches have an average compressive strengths of about 3,000 psi.

Table 12 - Compressive strength testing performed in 2006 by Kleinfelder

Sample Number	Length (in.)	Diameter (in.)	Description	Compressive Strength (psi)
S1-A	3.42	2.71	Span 1, south bound lane	4,030
S1-B	5.18	2.71	Span 1, north bound lane	3,270
S2-A	2.84	2.71	Span 2, north bound lane	4,920
S2-B	3.51	2.71	Span 2, south bound lane	2,900
S3-A	4.72	2.71	Span 3, north bound lane	2,430
S3-B	4.32	2.71	Span 3, south bound lane	3,200
S4-A	4.19	2.71	Span 4, south bound lane	4,470
S4-B	3.62	2.71	Span 4, north bound lane	3,800
S-E	5.46	2.71	South abutment, east side	2,480
S-W	5.45	2.71	South abutment, west side	1,920
N-E	5.29	2.71	North abutment, east side	3,220
N-W	5.20	2.71	North abutment, west side	2,920
RW	5.25	2.71	South retaining wall	2,420

Table 13 - Compressive strength testing performed in 2006 by Kleinfelder

Sample Number	Length (in.)	Diameter (in.)	Description	Compressive Strength (psi)
B1E-N	5.45	2.71	Bent 1, East Column, north side	2,850
B1W-N	3.87	2.71	Bent 1, West Column, north side	3,130
B3E-S	5.15	2.71	Bent 3, East Column, south side	2,020
B3W-S	6.74	2.71	Bent 3, West Column, south side	3,400

Each of the core locations selected, along with an image of the resulting sample can be seen in Figures 33 through 40. Complete images of all cores can be found in **Appendix 6**.

Figure 33 - Arch 1 East (Core 1B)

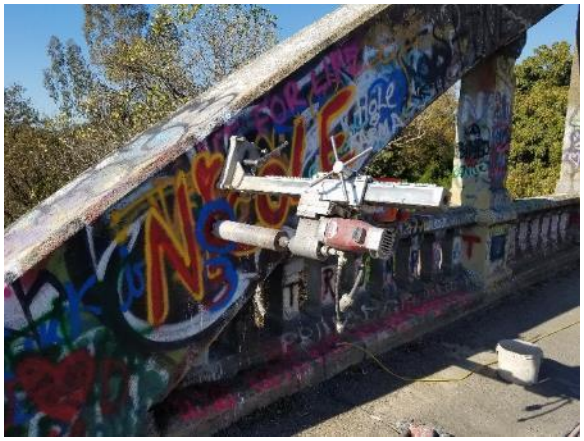


Figure 34 - Arch 3 West (Core 2)

Figure 35 - Arch 1 East extracted sample (Core 1B)

Figure 36 - Arch 3 West extracted sample (Core 2)

Figure 37 - Bridge Deck at Pier 4/Abut 5 (Core 3)

Figure 38 - West Girder (Core 4)

Figure 39 -Bridge Deck at Pier 4/Abutment 5 (Core 3)

Figure 40 - West girder extracted sample (core 4)

CONCLUSION

Below is a summary of observations from visual, borescope, GPR, and concrete testing.

Visual:

Based on a review of various elements of the structure, several areas were identified on the superstructure where spalling or exposed reinforcement were encountered. The majority of these areas will require repairs by removing unsound material, patching, and in some instances fiber wrapping to restore members to as-built condition. At the locations of major transverse cracks, if it is expected that no additional settlement will occur at the abutments, limited deck replacement with girder repairs shall be performed as outlined above. If further settlement can be anticipated, a concrete pile system and slope stability mitigation measures should be evaluated as part of the design.

For areas of the soffit, there are several areas where spalling and exposed reinforcement were observed. The majority of these areas must be repaired by removing unsound material and patching. Corresponding section details the repair strategy needed for different category of defects.

Borescope:

As observed from borescope images of reinforcement and due to lack of cracking around reinforcement, it appears that non-exposed rebar is in fair condition without excessive corrosion. No cracks associated with expansion of rebar, due to corrosion, was observed at the investigated locations. Based on these limited observations, no additional changes to the repair strategies were made.

Ground Penetrating Radar:

GPR was performed to locate reinforcement to aid with borescope and concrete coring. GPR scans indicate that the approximate rebar patterns are observed to be consistent with available As-Built drawings.

Review of the post-processed data for the small locations, including the arches, hangers, and pier column note various findings. Various locations exhibited possible material variation at varying depths. There may also be other objects, such as plates, air, pour lines or joints – that may result in the observed material variations. As the deck repair addresses the affected area scanned and because there are no signs of cracking at the other surveyed surfaces, no additional repair strategies are recommended.

Concrete Compressive Strength:

The compressive strength of concrete cores taken from the arches were between 2,800 psi and 3,600 psi. The bridge deck near Abutment 5 exhibited a compressive strength of 3,490 psi. The west girder at Abutment 1 appeared to have the highest compressive strength of 3,670 psi. Locations that were similar to those tested previously in 2006 were found to be within 10% of each other. In comparison with the data from the report produced previously, it appears that compressive strength at similar testing locations has generally maintained the same since the last compressive strength testing was performed.

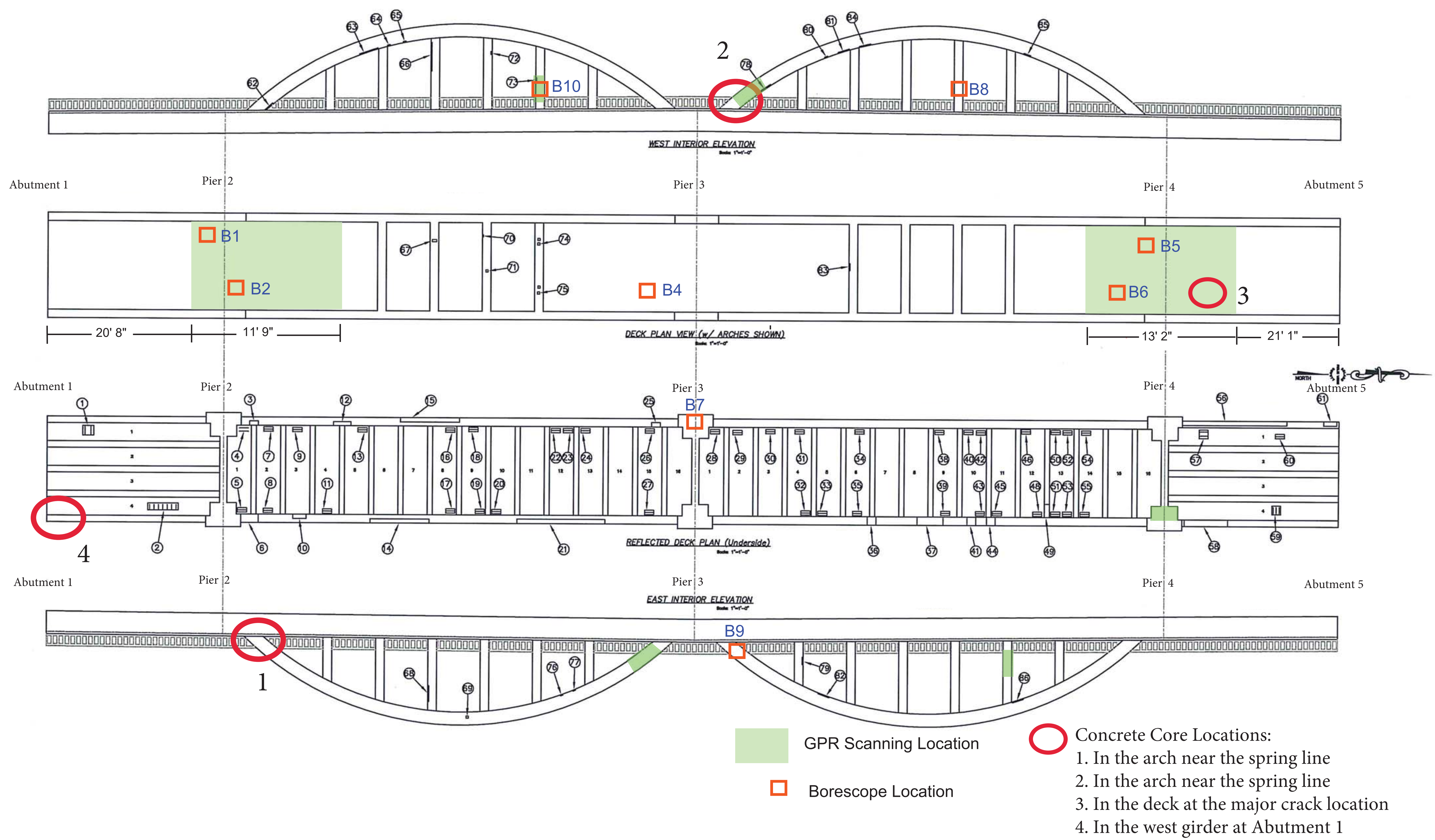
In regards to design analysis, based on testing performed on concrete cores, for sound or fully restored concrete the bridge deck and girders could be assumed at average compressive strength of 3,500 psi. The arches could be assumed at average compressive strengths of 3,000 psi.

If the above repair strategies are implemented, the investigated structural elements can be restored to the As-Built condition with the specified compressive strength.

Jinesh Mehta, P.E.
Technical Specialist
Alta Vista Solutions, Inc.

Jennifer Olarte
Engineer II
Alta Vista Solutions, Inc.

APPENDIX 1 – Work Location Diagram



WEST INTERIOR ELEVATION
B10
B8
2
Abutment 1
Pier 2
Pier 3
Pier 4
Abutment 5
B1
B2
B4
B5
B6
3
20' 8"
11' 9"
DECK PLAN VIEW (w/ ARCHES SHOWN)
13' 2"
21' 1"
B7
4
REFLECTED DECK PLAN (Underside)
EAST INTERIOR ELEVATION
B9
1
GPR Scanning Location
Borescope Location
Concrete Core Locations:
1. In the arch near the spring line
2. In the arch near the spring line
3. In the deck at the major crack location
4. In the west girder at Abutment 1

Appendix 2 – Visual Observations - Soffit

SPAN 1

Element	Photo #
Girder A	187-203
Bay 1	204-218
Girder B	219-234
Bay 2	234-254
Girder C	255-273
Bay 3	274-285
Girder D	286-300
Bay 4	301-316
Girder E	301-316
Girder Faces Defects	317-329

SPAN 2

Element	Photo #
Bay 16	442-443
Bay 15	444-445
Bay 14	446-447
Bay 13	448-449
Bay 12	450-453
Bay 11	454-455
Bay 10	456-457
Bay 1	458-459
Bay 2	460-461
Bay 3	462-463
Bay 4	464-465
Bay 5	466-467
Bay 6	468-469
Bay 7	470-471
Bay 8	472-473

SPAN 3

Element	Photo #
Bay 16	402-404
Bay 15	405-407
Bay 14	408-411
Bay 13	412-414
Bay 12	415-417
Bay 11	418-419
Bay 10	420-421
Bay 9	422-423
Bay 8	424-425
Bay 7	426-227
Bay 6	428-429
Bay 5	430-431
Bay 4	432-433
Bay 3	434-435
Bay 2	436-437
Bay 1	438-439
Girder Faces Defects	440-441

SPAN 4

Element	Photo #
Bay 4	330-345
Bay 3	346-358
Bay 2	359-372
Bay 1	373-386
Girder Faces Defects	387-401

COLOR KEY

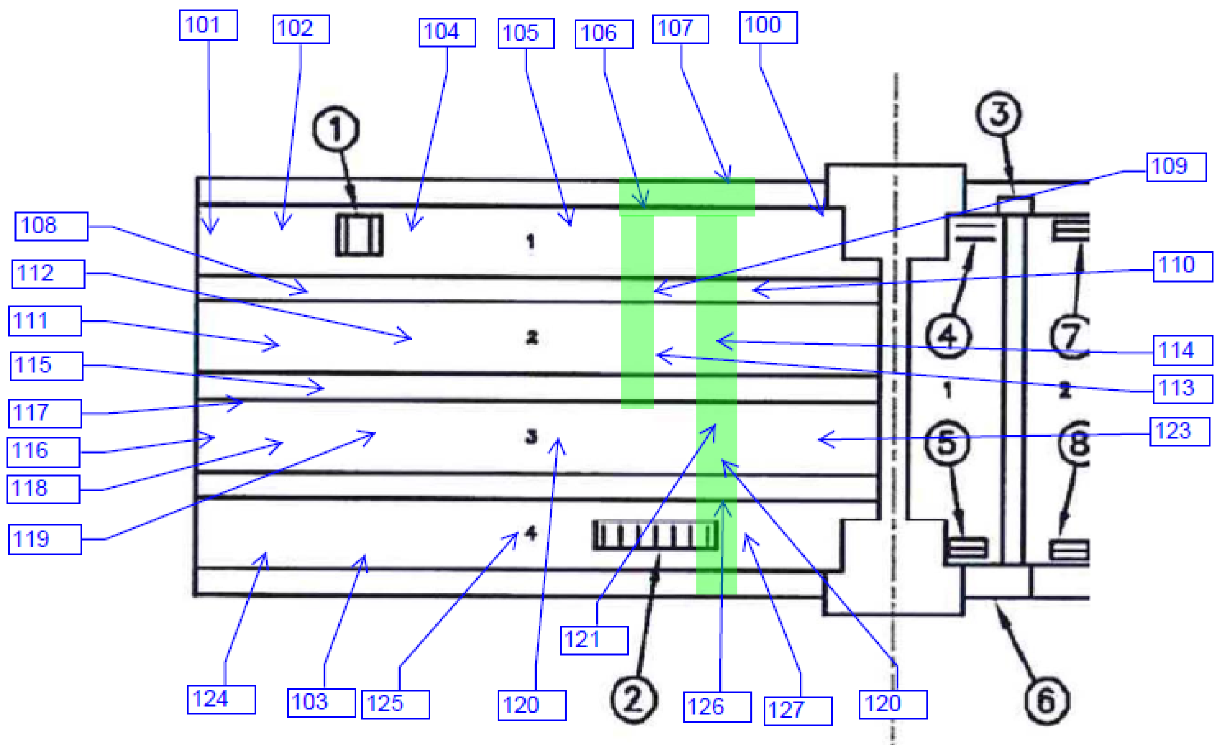
Category	
1	Good
2	Fair
3	Poor
4	Severe

SPAN 1

Span	Defect Number	Description	Photo Number	Category	Area (sq.ft)
Span I	100	Rock Pocket - End of Girder 1 and Bent 2	219	2	1
Span I	-	No Visible Defects	187-203	1	0
Span I	101	Deck Drain, Longitudinal and Transverse Cracking, Visible Rust, Effloresce	204-205	4	9
Span I	102	Deck Drain, Transverse Cracking, Effloresce	206-208	4	9
Span I	103	Exposed Rebar, Effloresce	209	3	2
Span I	104	Deck Drain, Spall, Exposed Rebar	210	3	2
Span I	105	Deck Drain, Spall, Exposed Rebar	211-213	3	3
Span I	106	Major Transverse Crack, Exposed Rebar, Spalling	214	4	3
Span I	107	Major Transverse Crack	215-217	4	7
Span I	-	Effloresce from Deck Drain	218-220	1	0
Span I	-	No Visible Defects	221-223	1	0
Span I	108	Minor Transverse Crack	224	2	3
Span I	-	No Visible Defects	225-228	1	0
Span I	109	Major Longitudinal Crack - 8ft Running North - West of Girder	229-230	4	18
Span I	110	Minor Longitudinal Crack - East and West of Girder	231-232	2	8
Span I	111	Minor Crack	233	2	15
Span I	-	No Visible Defects	234-236	1	0
Span I	-	No Visible Defects	237-240	1	0
Span I	112	Square Void - Closer to Girder B	241-244	2	4
Span I	-	No Visible Defects	245-246	1	0
Span I	113	Deck Drain – Possible piece of Wood?	247-248	2	1
Span I	114	Major Transverse Crack, Effloresce, Void near Girder B	249-253	4	13

Span I	-	No Visible Defects	254-256	1	0
Span I	-	No Visible Defects	257-258	1	0
Span I	115	Surface/Mud Cracking on Girder Soffit	259-260	2	9
Span I	-	No Visible Defects - Soffit of Girder Only	261-273	1	0
Span I	116	Minor Transverse Crack	274	2	6
Span I	117	Void West of Bay	275	2	5
Span I	118	Minor Transverse Crack	276-277	2	7
Span I	119	Void Middle of the Bay	278-280	3	6
Span I	120	Minor Transverse Crack	281	2	7
Span I	121	Major Crack, Visible Rust, Effloresce	282	4	11
Span I	122	Major Crack	283	4	2
Span I	123	Major Transverse Crack, Effloresce	284-286	4	25
Span I	-	No Visible Defects	287	1	0
Span I	-	No Visible Defects - Soffit of Girder Only	288-302	1	0
Span I	124	Void	303-308	3	31
Span I	125	2 Voids, Exposed Rebar	309-311	4	12
Span I	126	Major Cracking, Exposed Rebar	312-313	4	6
Span I	127	Major Transverse Crack, Effloresce, Void near Girder B	314-317	4	6
Span I	-	Effloresce	318	2	0
Span I	128	Girder A - Major Vertical Crack - Interior Face	319	4	3
Span I	129	Girder B - West Face - Major Vertical Crack	320	4	4
Span I	130	Girder B - East Face - Major Vertical Crack, Effloresce	321	4	4
Span I	131	Girder C - West Face - Major Vertical Crack	322	4	4
Span I	132	Girder C - East Face - Major Vertical Crack	323	4	3
Span I	133	Girder D - West Face - 2 Major Vertical Cracks	324	4	3
Span I	134	Girder D - East Face - 2 Major Vertical Cracks (3ft apart)	325	4	9
Span I	135	Girder E - Interior Face -2 Major Vertical Cracks	326-329	4	5

SPAN 1



Abutment 1

Pier 2

Span 1


Figure 1 - Defect 100 (DSC_0219)


Figure 2 - Defect 101 (DSC_0204)


Figure 3 - Defect 102 (DSC_0206)


Figure 4 - Defect 103 (DSC_0209)


Figure 5 - Defect 104 (DSC_0210)


Figure 6 - Defect 105 (DSC_0212)

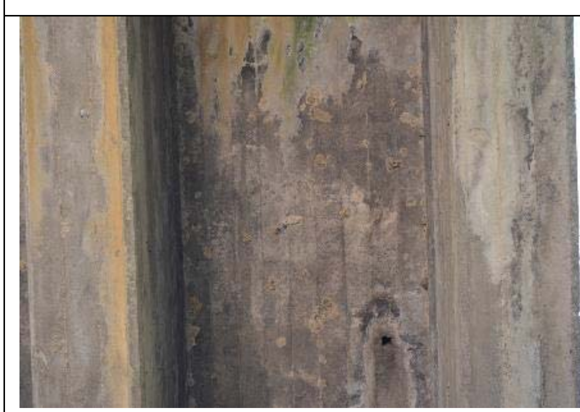


Figure 7 - Defect 106 (DSC_0214)


Figure 8 - Defect 107 (DSC_0216)

Span 1

Figure 9 - Defect 108 (DSC_0224)

Figure 10 - Defect 109 (DSC_0229)

Figure 11 - Defect 110 (DSC_0232)

Figure 12 - Defect 111 (DSC_0233)

Figure 13 - Defect 112 (DSC_0242)

Figure 14 - Defect 113 (DSC_0248)

Figure 15 - Defect 114 (DSC_0250)

Figure 16 - Defect 115 (DSC_0259)

Span 1


Figure 17 - Defect 116 (DSC_0274)


Figure 18 - Defect 117 (DSC_0275)


Figure 19 - Defect 118 (DSC_0277)


Figure 20 - Defect 119 (DSC_0279)


Figure 21 - Defect 120 (DSC_0281)


Figure 22 - Defect 121 (DSC_0282)


Figure 23 – Defect 122 (DSC_0283)


Figure 24 - Defect 123 (DSC_0284)

Span 1


Figure 25 - Defect 124 (DSC_0307)


Figure 26 - Defect 125 (DSC_0311)


Figure 27 - Defect 126 (DSC_0313)


Figure 28 - Defect 127 (DSC_0315)


Figure 29 – Defect 128 (DSC_319)


Figure 30 – Defect 129 (DSC_320)


Figure 31 – Defect 130 (DSC_321)

Figure 32 – Defect 131 (DSC_322)

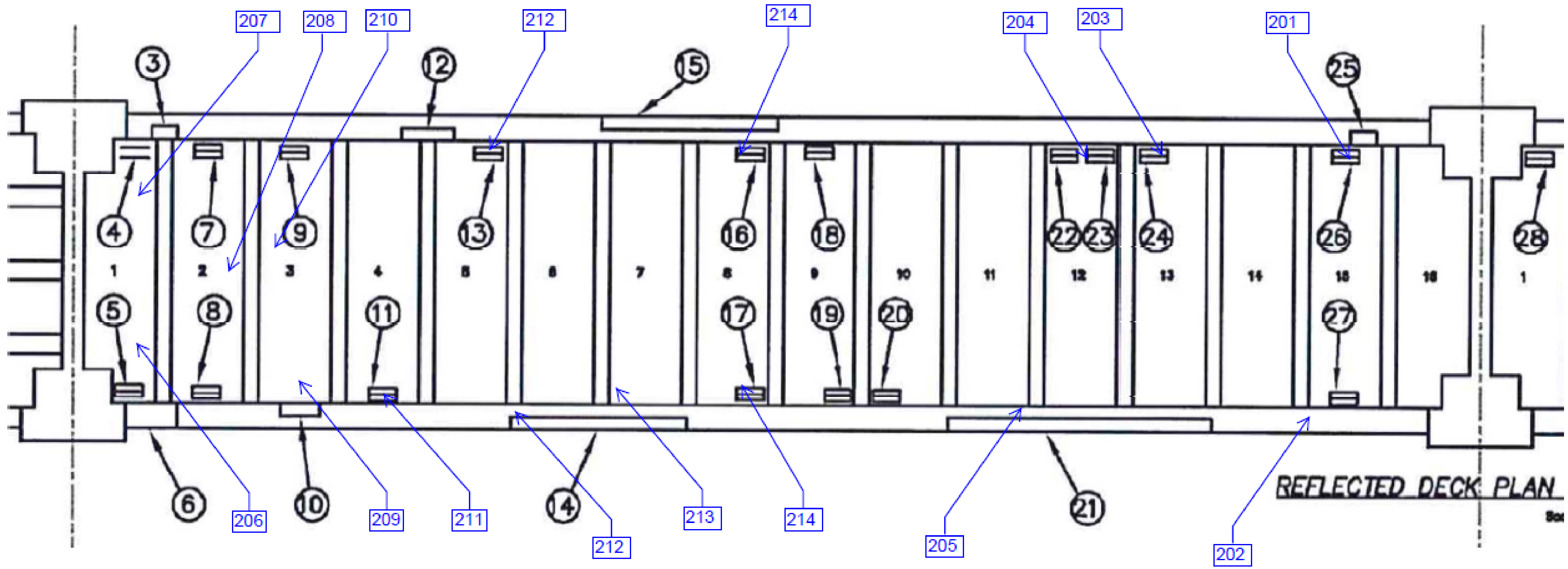
Span 1

 Figure 33 – Defect 132 (DSC_323)	 Figure 34 – Defect 133 (DSC_324)
 Figure 35 – Defect 134 (DSC_325)	 Figure 36 – Defect 135 (DSC_326)

SPAN 2

Span	Defect Number	Description	Photo Number	Category	Area (sq.ft)
Span II	-	Bay 16 - No Visible Defects	442-443	1	0
Span II	201	Bay 15 - Exposed Rebar	444	4	4
Span II	202	Bay 15 - Spalling on Girder	445	3	4
Span II	-	Bay 14 - No Visible Defects	446-447	1	0
Span II	203	Bay 13 - Exposed Rebar, Possible Void	448	3	14
Span II	-	Bay 13 - No Visible Defects	449	1	0
Span II	204	Bay 12 - Exposed Rebar, Possible Cracking	450-453	3	3
Span II	205	Bay 11 - Exposed Rebar on Girder and at Bottom of Photo	454-455	3	3
Span II	-	Bay 10 - No Visible Defects	456-457	1	0
Span II	206	Bay 1 - Exposed Rebar, Effloreces	458	3	9
Span II	207	Bay 1 - Exposed Rebar, Effloreces	459	4	7
Span II	208	Bay 2 - Exposed Rebar, Void, Various Suface Cracks	460-461	3	6
Span II	209	Bay 3 - Possible Void, Minor Longitudinal Cracking	462	2	5
Span II	210	Bay 3 - Minor Transverse Cracking	463	2	5
Span II	211	Bay 4 - Exposed Rebar	464	4	3
Span II	-	Bay 4 - No Visible Defects	465	1	0
Span II	212	Bay 5 - Exposed Rebar on Girder	466	4	2
Span II	-	Bay 5 - Exposed Rebar	467	3	2
Span II	-	Bay 6 - No Visible Defects	468-469	1	0
Span II	213	Bay 7 - Exposed Rebar	470-471	3	3
Span II	214	Bay 8 - Exposed Rebar	472-473	3	4
Span II	-	Bay 9 - No Visible Defects	474-477	1	0

SPAN 2



Pier 2

Pier 3

Span 2

 Figure 1 – Defect 201 (DSC_444)	 Figure 2 – Defect 202 (DSC_445)
 Figure 3 – Defect 203 (DSC_448)	 Figure 4 - Defect 204 (DSC_452)
 Figure 5 - Defect 205 (DSC_454)	 Figure 6 - Defect 206 (DSC_458)
 Figure 7 - Defect 207 (DSC_0459)	 Figure 8 - Defect 208 (DSC_0460)

Span 2

Figure 9 - Defect 209 (DSC_0462)

Figure 10 = Defect 210 (DSC_463)

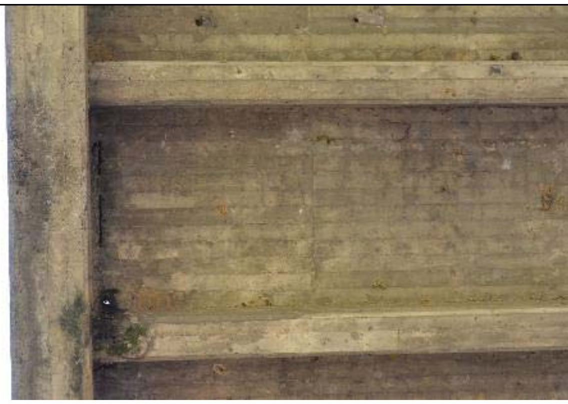


Figure 4 - Defect 211 (DSC_0464)

Figure 5 - Defect 212 (DSC_0466)

Figure 6 - Defect 213 (DSC_0470)

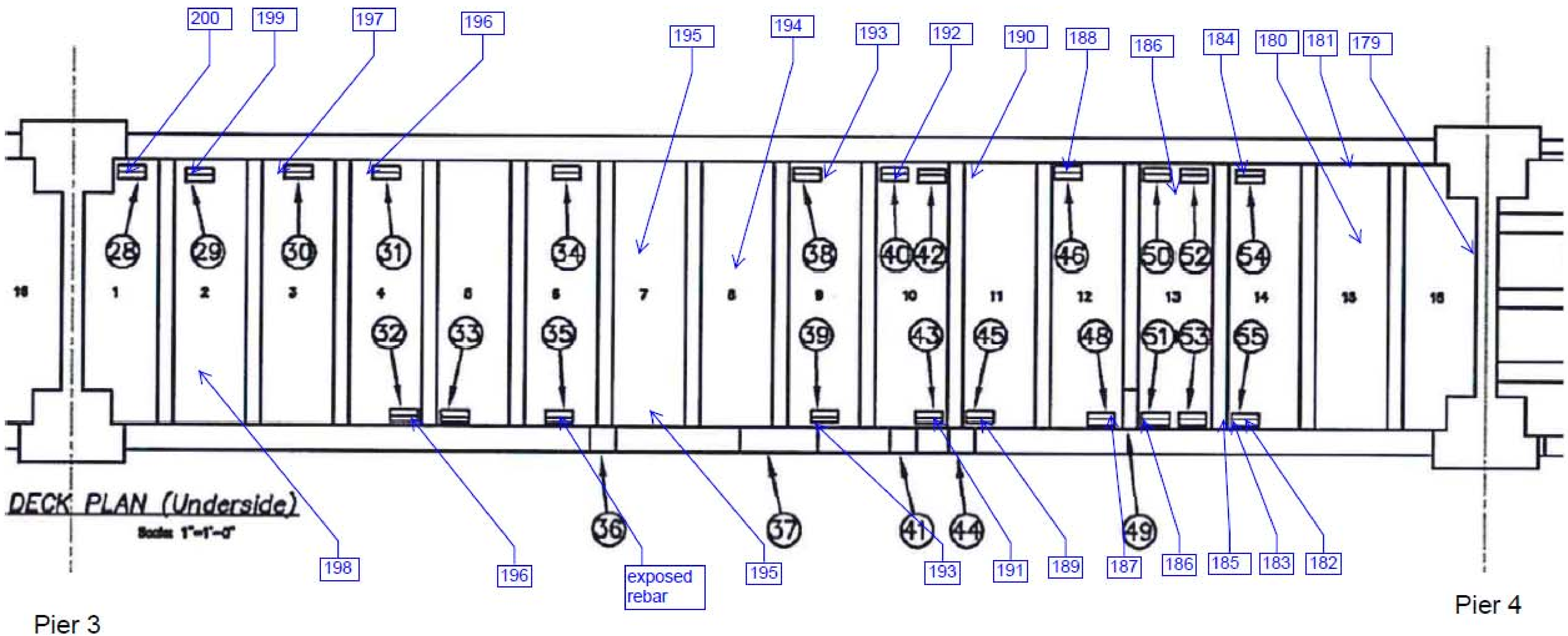
Figure 7 - Defect 214 (DSC_0472)

SPAN 3

Span	Defect Number	Description	Photo Number	Category	Area (sq.ft)
Span III	179	Bay 16 - Major Transverse Cracking, Void	402-404	4	4
Span III	180	Bay 15 - Minor Transverse Cracking, Void	405-406	3	6
Span III	181	Bay 15 - Exposed Rebar	407	3	2
Span III	182	Bay 14 - Exposed Rebar	408	4	5
Span III	183	Bay 14 - Exposed Rebar	409	4	5
Span III	184	Bay 14 - Exposed Rebar	410	4	3
Span III	185	Bay 14 - Exposed Rebar	411	4	8
Span III	186	Bay 13 - Major Spall, Major Exposed Rebar	412-414	4	5
Span III	187	Bay 12 - Exposed Rebar	415-416	4	7
Span III	188	Bay 12 - Void, Intermediate Cracking	417	3	2
Span III	189	Bay 11 - Exposed Rebar	418	4	3
Span III	190	Bay 11 - Exposed Rebar, Intermediate Transverse Cracking	419	3	2
Span III	191	Bay 10 - Exposed Rebar	420	4	3
Span III	192	Bay 10 - Exposed Rebar	421	3	2
Span III	193	Bay 9 - Exposed Rebar on Girder	422	4	3
Span III	194	Bay 8 - Intermediate Transverse Cracking	424-425	3	5
Span III	195	Bay 7 - Exposed Rebar, Intermediate Transverse Cracking	426-427	3	21
Span III	-	Bay 6 - Exposed Rebar	428	4	2
Span III	-	No Visable Defects	429-431	2	6
Span III	196	Bay 4 - Exposed Rebar	432-433	3	4
Span III	-	No Visable Defects	434	1	0
Span III	197	Bay 3 - Exposed Rebar	435	4	4
Span III	198	Bay 2 - Transverse Cracking	436	2	4

Span III	199	Bay 2 - Exposed Rebar	437	4	4
Span III	-	No Visable Defects	438	1	0
Span III	200	Bay 1 - Exposed Rebar	439	4	4
Span III	-	No Visable Defects	440-441	1	0

SPAN 3



200
199
197
196
195
194
193
192
190
188
186
184
180
181
179
DECK PLAN (Underside)
198
196
exposed rebar
195
193
191
189
187
186
185
183
182
Pier 3
Pier 4

Span 3

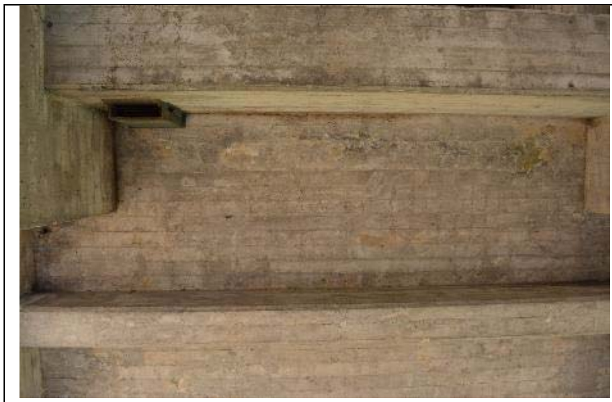


Figure 1 - Defect 179 (DSC_0403)


Figure 2 - Defect 180 (DSC_0406)


Figure 3 - Defect 181 (DSC_0407)


Figure 4 - Defect 182 (DSC_0408)


Figure 5 - Defect 183 (DSC_0409)


Figure 6 - Defect 184 (DSC_0410)


Figure 7 - Defect 185 (DSC_0411)


Figure 8 - Defect 186 (DSC_0412)

Span 3


Figure 9 - Defect 187 (DSC_0415)


Figure 10 - Defect 188 (DSC_0417)


Figure 11 - Defect 189 (DSC_0418)


Figure 12 - Defect 190 (DSC_419)


Figure 13 - Defect 191 (DSC_0420)

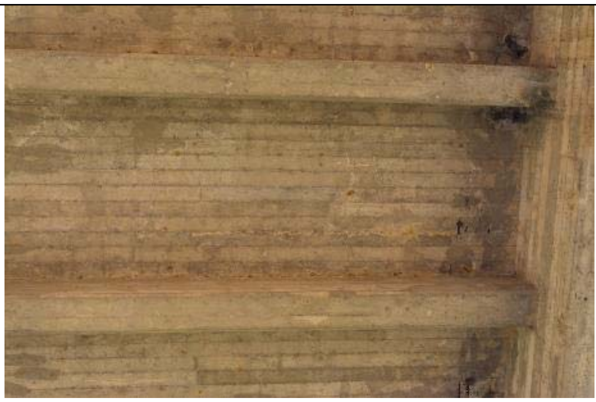


Figure 14 - Defect 192 (DSC_0421)


Figure 15 - Defect 193 (DSC_0422)

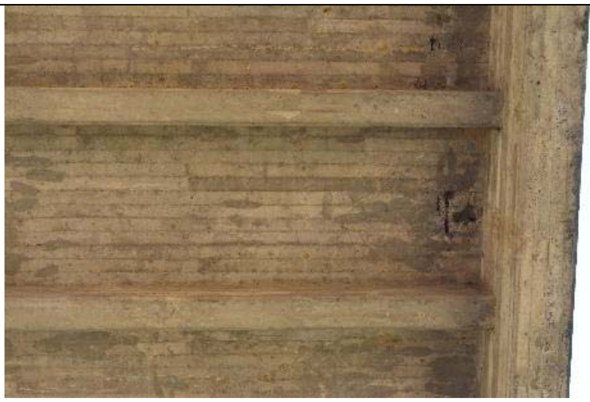


Figure 16 - Defect 193 (DSC_0423)

Span 3


Figure 17 - Defect 194 (DSC_0425)


Figure 18 - Defect 195 (DSC_0426)


Figure 19 – Unnumbered Defect (DSC_428) Bay 6

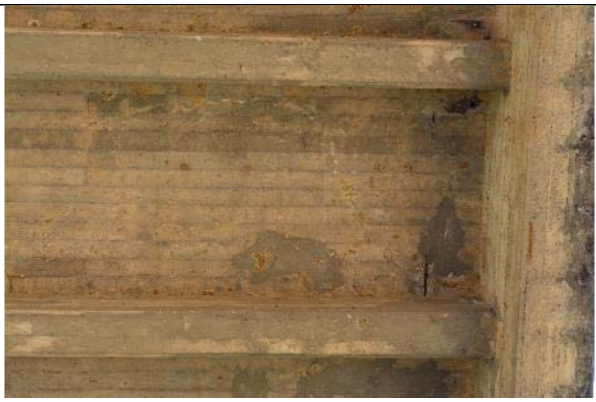


Figure 20 - Defect 196 (DSC_0433)


Figure 21 - Defect 197 (DSC_0435)


Figure 22 - Defect 198 (DSC_0436)


Figure 23 - Defect 199 (DSC_0437)

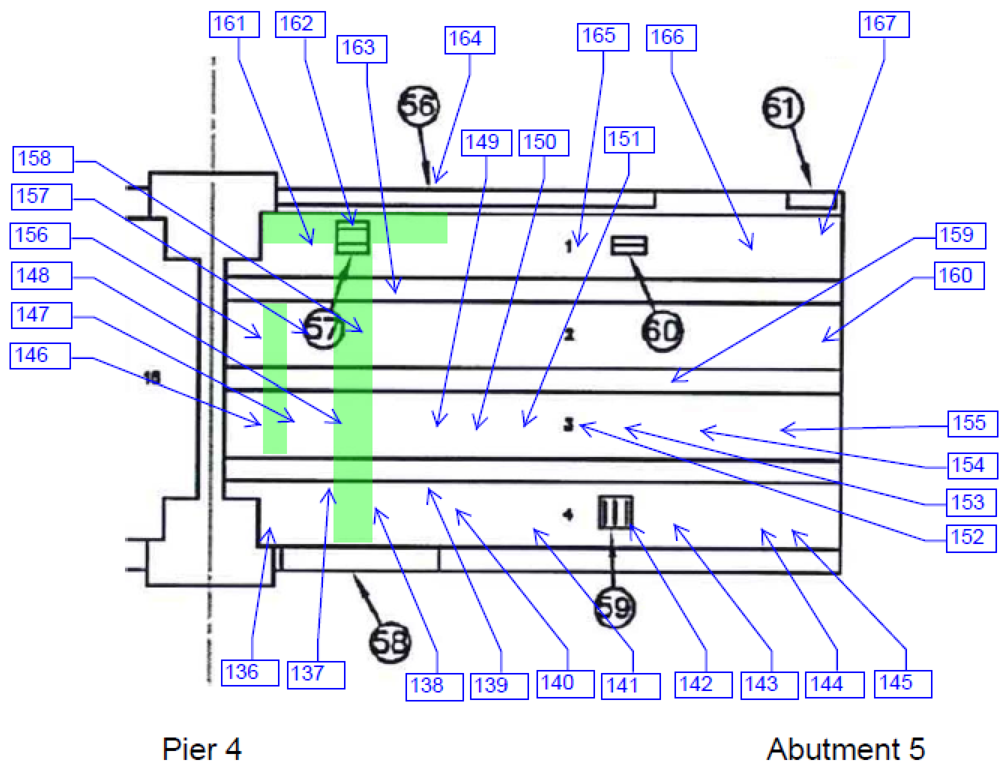

Figure 24 - Defect 200 (DSC_0439)

SPAN 4

Span	Defect Number	Description	Photo Number	Category	Area (sq.ft)
Span IV	136	Deck Drain, Exposes Rebar	330	4	10
Span IV	137	Pour Consolidation/Rock Pockets	331	3	26
Span IV	138	Major Transverse Crack, Exposed Rebar, Effloresce	332	4	51
Span IV	139	Major Transverse Crack, Exposed Rebar, Effloresce	333	4	51
Span IV	140	Exposed Rebar, Spalling	334-335	4	52
Span IV	141	Void, Effloresce	336-337	2	3
Span IV	142	Exposed Rebar	338-339	3	36
Span IV	-	Not Used	340-341	1	0
Span IV	143	Void	342-343	3	3
Span IV	144	Minor Surface Crack	344	2	1
Span IV	145	Minor Transverse Cracking, Effloresce	345	2	6
Span IV	146	Major Transverse Crack	346	3	11
Span IV	147	Surface Cracks in all Directions	347-348	2	17
Span IV	148	Major Transverse Crack	349	4	11
Span IV	149	Surface Cracks in all Directions, Major Transverse Crack	350	3	11
Span IV	150	Surface Cracks in all Directions	351-352	2	2
Span IV	151	Surface Cracks Mostly Longitudinal	353-354	2	5
Span IV	152	Surface Cracks in all Directions	355	2	7
Span IV	153	Surface Cracks in all Directions	356	2	4
Span IV	154	Major Longitudinal Crack	357	3	11
Span IV	155	Various Cracks in all Directions	358	2	9
Span IV	156	Major Transverse Crack, Void	359-360	4	6
Span IV	157	Various Cracks in all Directions, Void	361	2	11
Span IV	158	Major Transverse Crack, Exposed Rebar, Spalling	362-363	4	9

Span IV	-	No Visable Defects	364-369	1	0
Span IV	159	Mud Cracking on Girder C	370	2	5
Span IV	-	No Visable Defects	371	1	0
Span IV	160	Various Cracking	372	2	7
Span IV	161	Minor Longitudinal Cracking, Void	373-374	2	15
Span IV	162	Major Transverse Crack, Exposed Rebar, Spalling	375-376	4	17
Span IV	163	Major Longitudinal Cracking	377	4	17
Span IV	164	Major Longitudinal Cracking	378	4	10
Span IV	165	Major Transverse Crack on Girder, Voids	379-381	4	9
Span IV	-	No Visable Defects	382	1	0
Span IV	166	Major Transverse Crack, Void	383	3	8
Span IV	-	No Visable Defects	384	1	0
Span IV	167	Pour Consolidation, Exposed Rebar	385-386	4	6
Span IV	168	Girder E - Interior Face - Major Vertical Crack	387	4	10
Span IV	169	Girder D - West Face - Major Vertical Crack	388	4	3
Span IV	170	Girder C - West Face - Major Vertical Crack	389	3	2
Span IV	171	Girder B - West Face - 2 Major Vertical Cracks	390	4	2
Span IV	172	Girder A - East Face - Major Vertical Crack	391	4	6
Span IV	173	Girder B - East Face -2 Major Vertical Cracks	392	3	2
Span IV	174	Girder B - East Face - Intermediate Cracking	393	3	3
Span IV	175	Girder C - East Face - 3 Major Vertical Cracks	394-395	3	2
Span IV	176	Girder D - East Face - Major Vertical Crack	396	4	2
Span IV	177	Girder D - East Face - Intermediate Cracking	397	3	2
Span IV	178	Horizontal Cracking at Abuttment Base (all the way through)	398-401	4	2

SPAN 4


161
162
163
164
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166
167
149
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151
158
157
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148
147
146
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160
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154
153
152
136
137
138
139
140
141
142
143
144
145
56
57
58
59
60
61
1
2
3
4
Pier 4
Abutment 5

Span 4

 *Figure 1 - Defect 136 (DSC_0330)*	 *Figure 2 - Defect 137 (DSC_0331)*
 *Figure 3 - Defect 138 (DSC_0332)*	 *Figure 4 - Defect 139 (DSC_0333)*
 *Figure 5 - Defect 140 (DSC_0334)*	 *Figure 6 - Defect 141 (DSC_0336)*
 *Figure 7 - Defect 142 (DSC_0339)*	 *Figure 8 - Defect 143 (DSC_0343)*

Span 4


Figure 9 - Defect 144 (DSC_0344)


Figure 10 - Defect 145 (DSC_0345)


Figure 11 - Defect 146 (DSC_0346)


Figure 12 - Defect 147 (DSC_0348)


Figure 13 - Defect 148 (DSC_0349)


Figure 14 - Defect 149 (DSC_0350)


Figure 15 - Defect 150 (DSC_0352)


Figure 16 - Defect 151 (DSC_0353)

Span 4


Figure 17 - Defect 152 (DSC_0355)


Figure 18 - Defect 153 (DSC_0356)


Figure 19 - Defect 154 (DSC_0357)


Figure 20 - Defect 155 (DSC_0358)


Figure 21 - Defect 156 (DSC_0359)


Figure 22 - Defect 157 (DSC_0361)


Figure 23 - Defect 158 (DSC_0362)


Figure 24 - Defect 159 (DSC_0370)

Span 4

Figure 25 - Defect 160 (DSC_0372)

Figure 26 - Defect 161 (DSC_0374)

Figure 27 - Defect 162 (DSC_0375)

Figure 28 - Defect 163 (DSC_0377)

Figure 29 - Defect 164 (DSC_0378)

Figure 30 - Defect 165 (DSC_0380)

Figure 31 - Defect 166 (DSC_0383)

Figure 32 - Defect 167 (DSC_0386)

Span 4

 *Figure 33 – Defect 168 (DSC_387)*	 *Figure 34 – Defect 169 (DSC_388)*
 *Figure 15 – Defect 170 (DSC_389)*	 *Figure 36 – Defect 171 (DSC_390)*
 *Figure 37 – Defect 172 (DSC_391)*	 *Figure 38 – Defect 173 (DSC_392)*
 *Figure 39 – Defect 174 (DSC_393)*	 *Figure 40 – Defect 175 (DSC_394)*

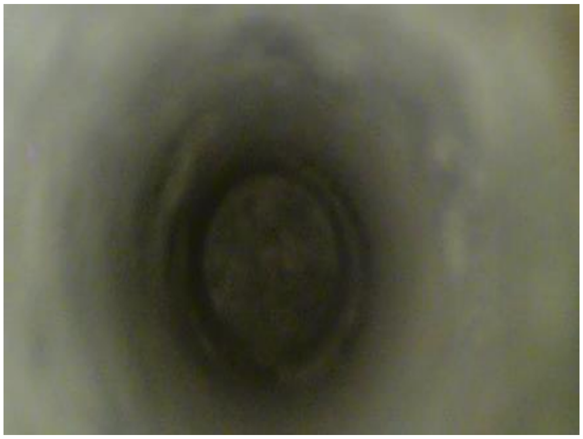
Span 4


Figure 41 – Defect 176 (DSC_396)

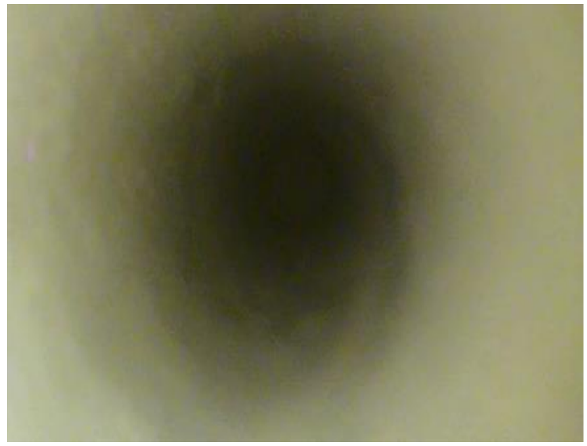

Figure 42 – Defect 177 (DSC_397)


Figure 23 – Defect 178 (DSC_399)

APPENDIX 3 – Borescope Images



Hole B1

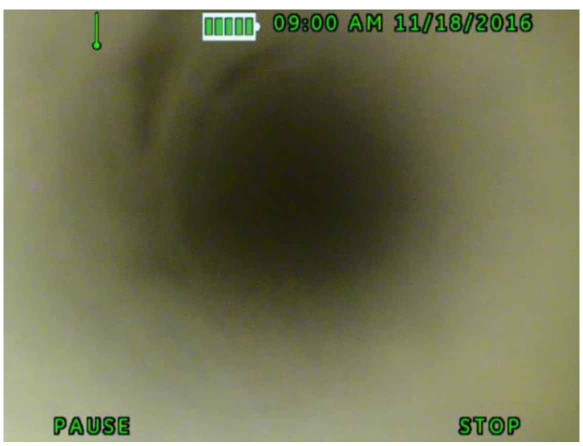



09:41 AM 11/18/2016
B6


10:50 AM 11/18/2016
B7


02:32 PM 11/18/2016
B8


B9

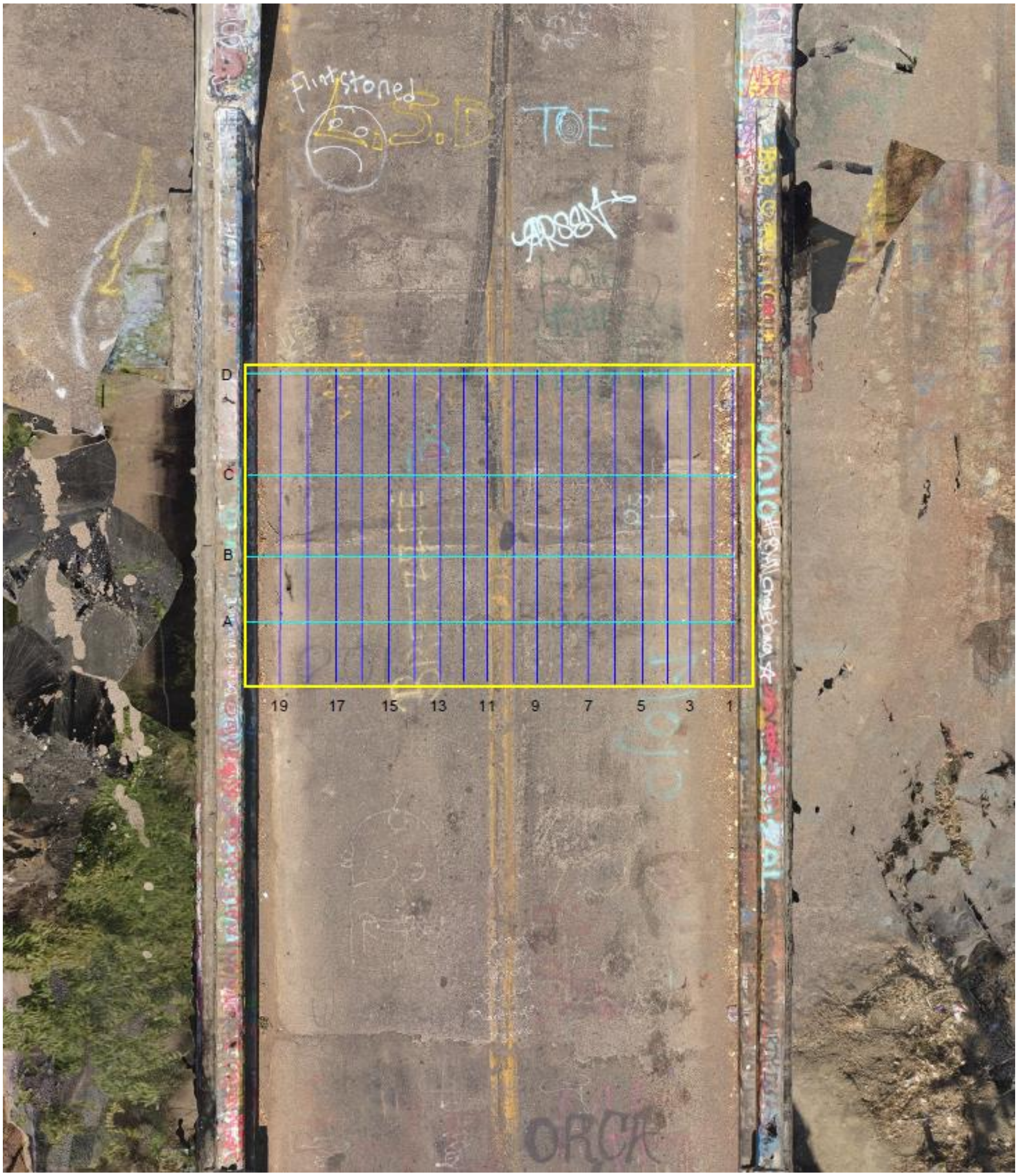

09:00 AM 11/18/2016
PAUSE
STOP

Hole B9

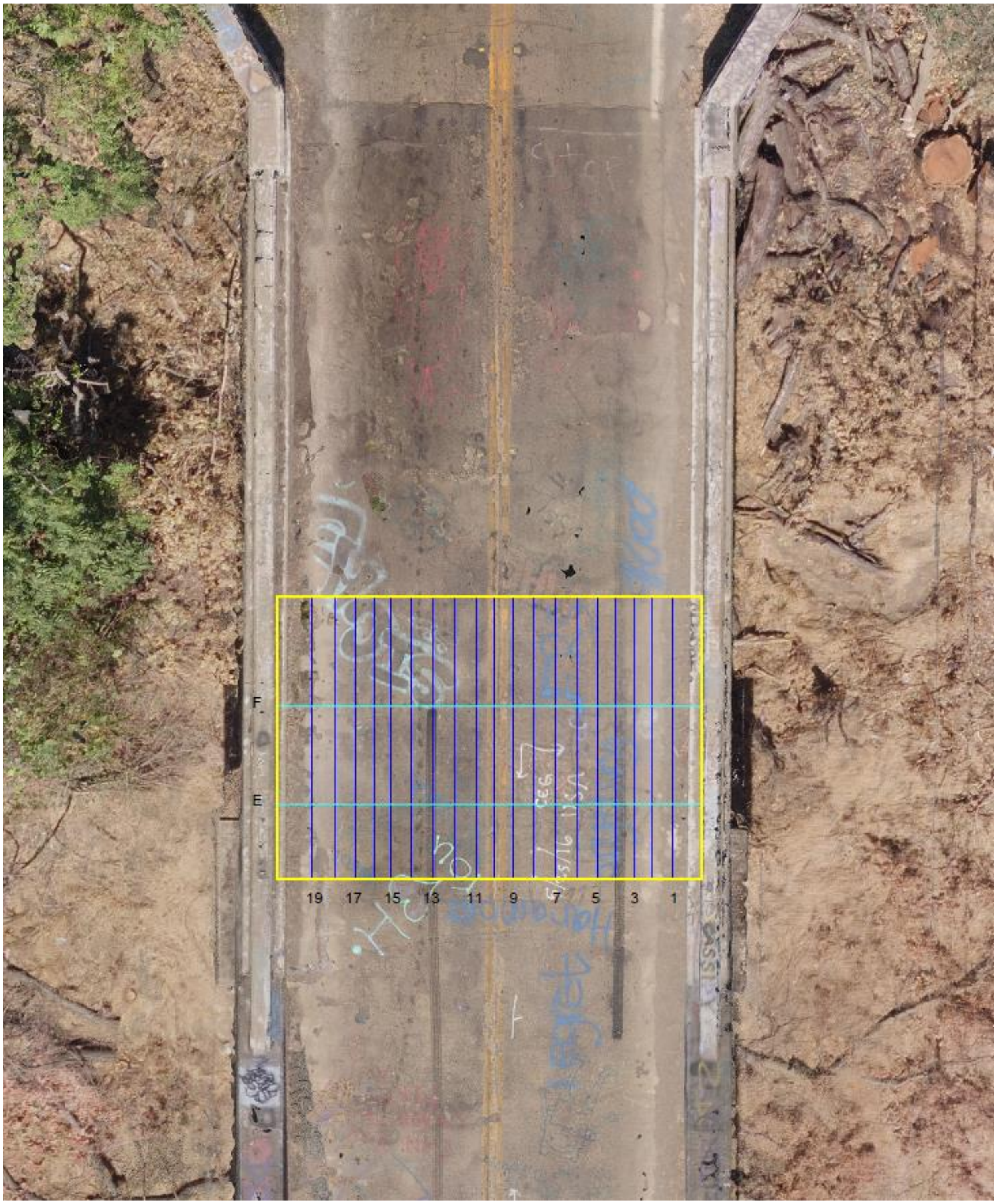

02:59 PM 11/18/2016
B10

Appendix 4 – GPR Scan Areas

Abutment 1



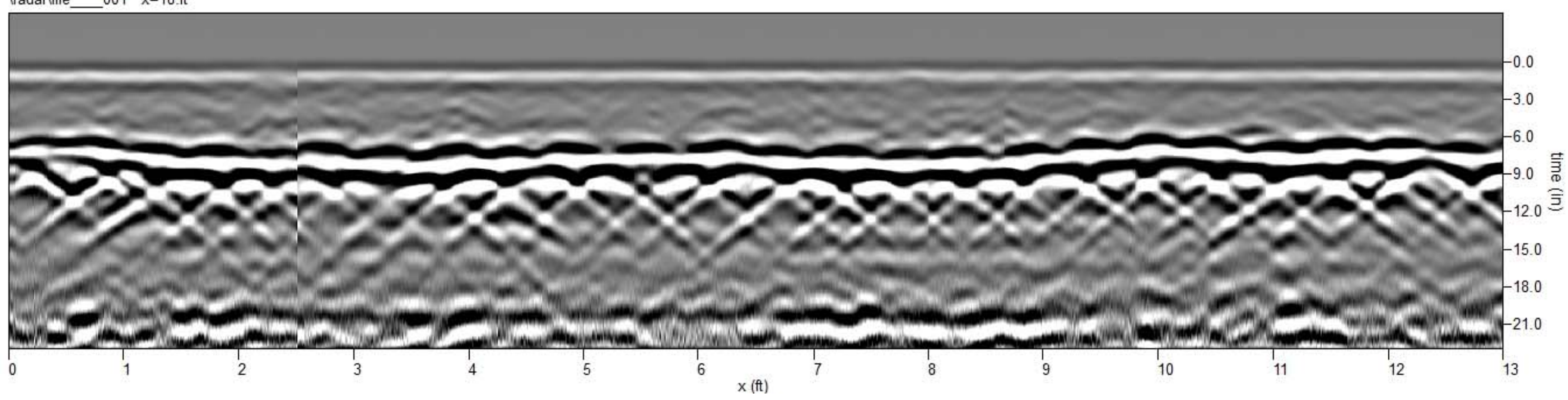
Abutment 5



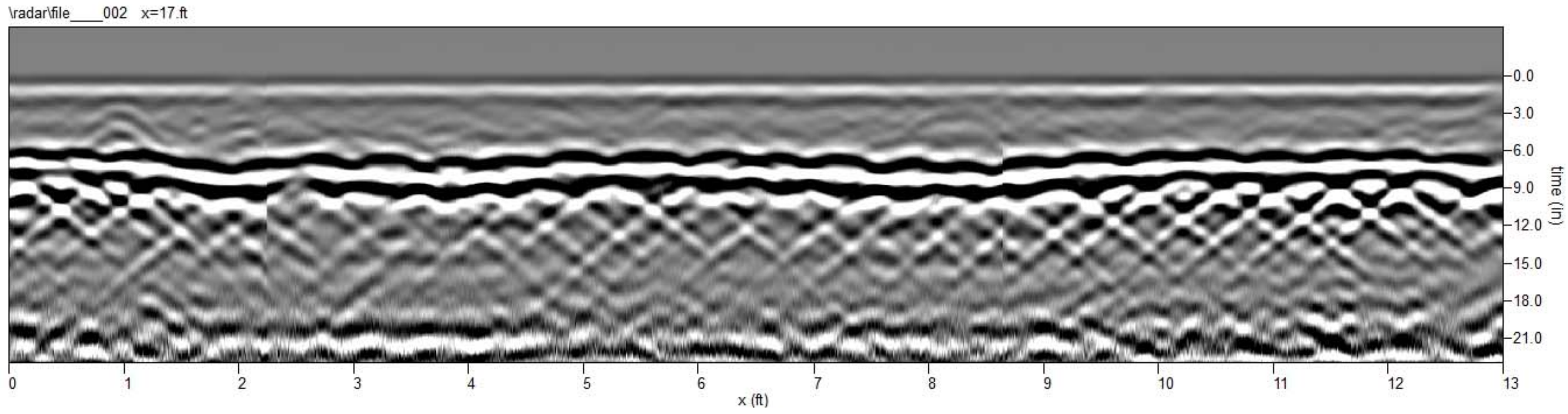
Appendix 5 – GPR Scans

ABUTMENT 1 – LONGIDUTINAL SCANS

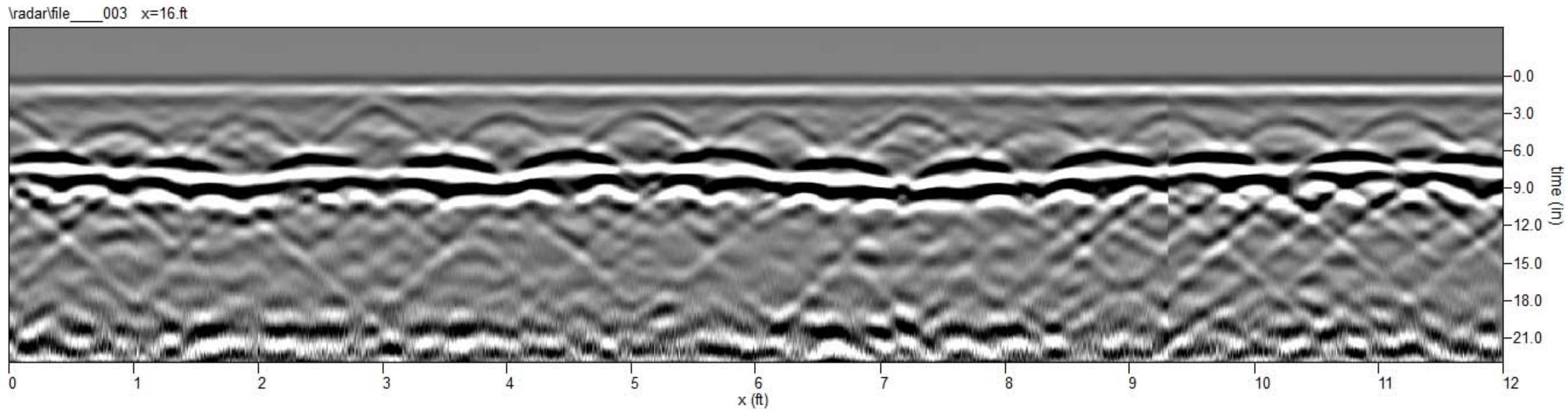
\radar\file____001   x=18.ft



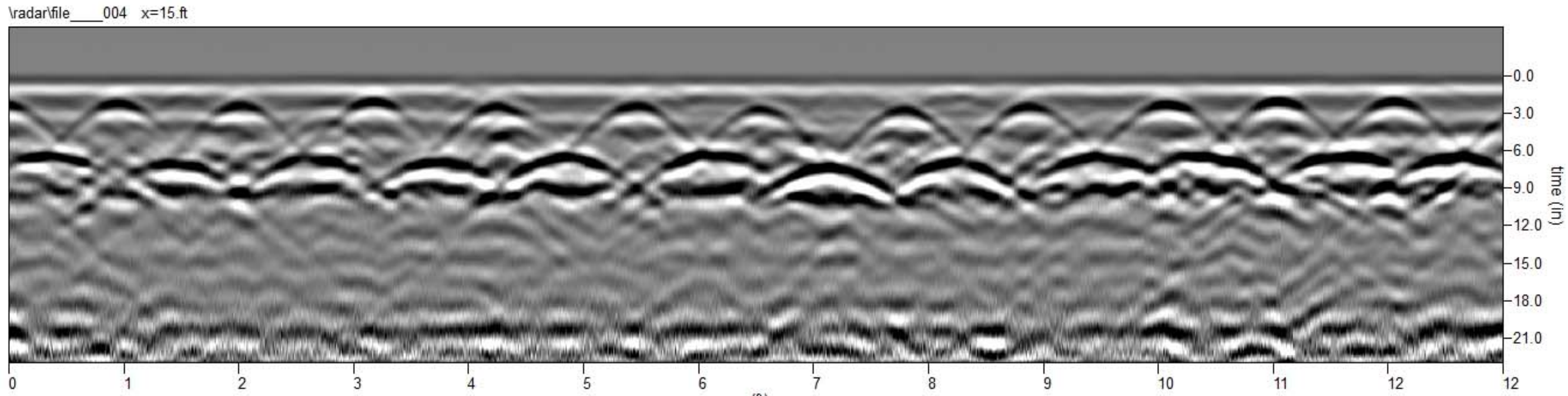
\radar\file____002   x=17.ft



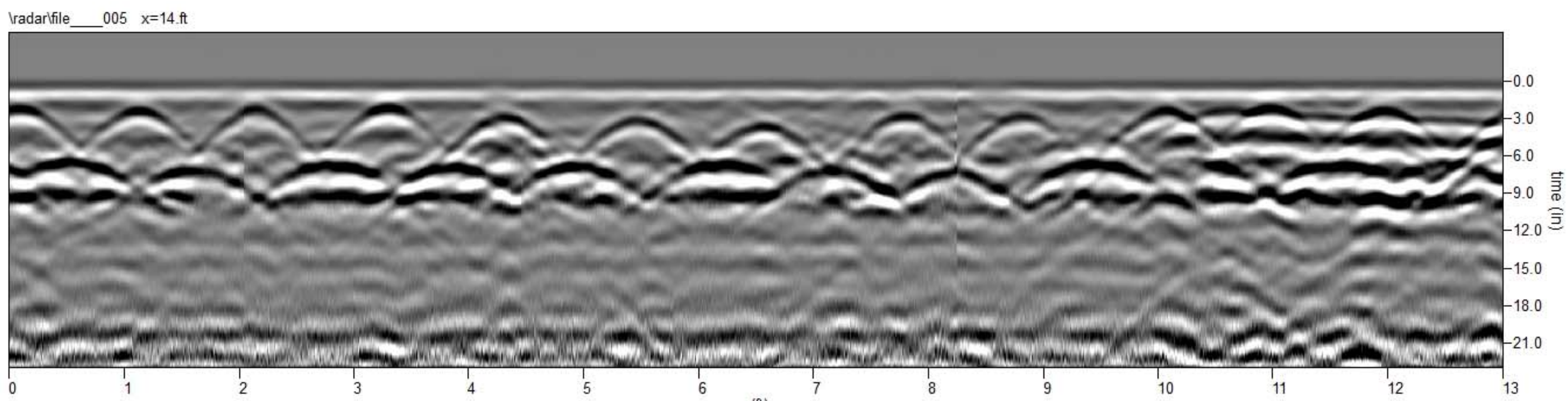
\radar\file____003   x=16.ft



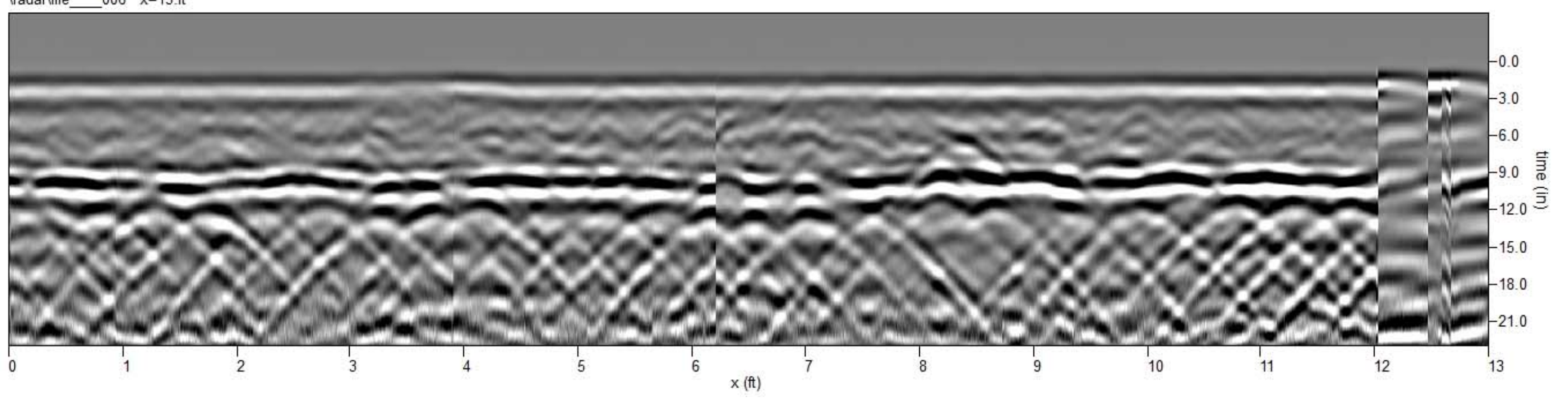
\radar\file____004   x=15.ft



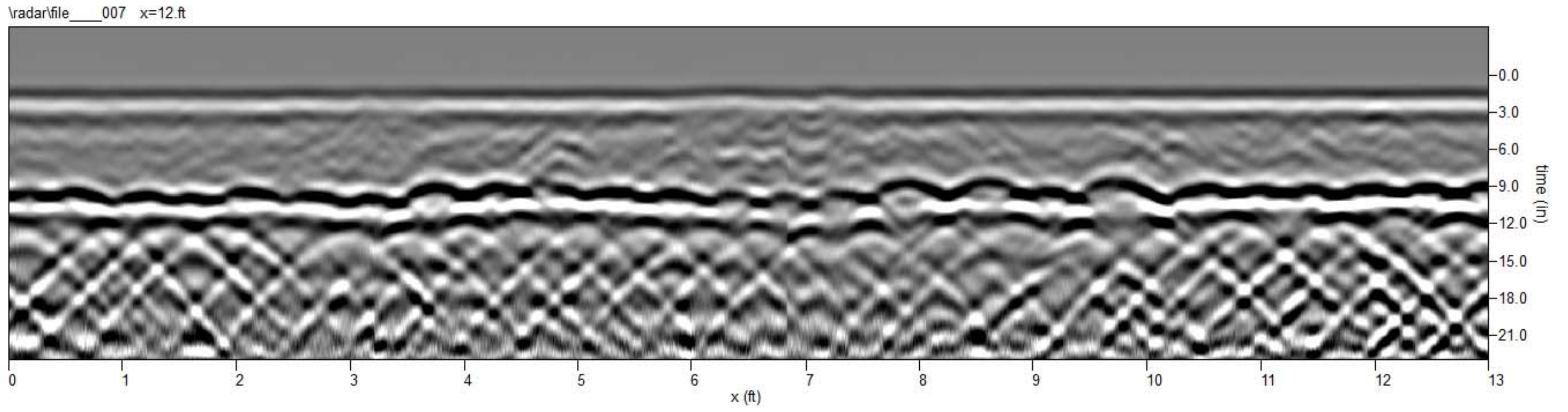
\radar\file____005   x=14.ft



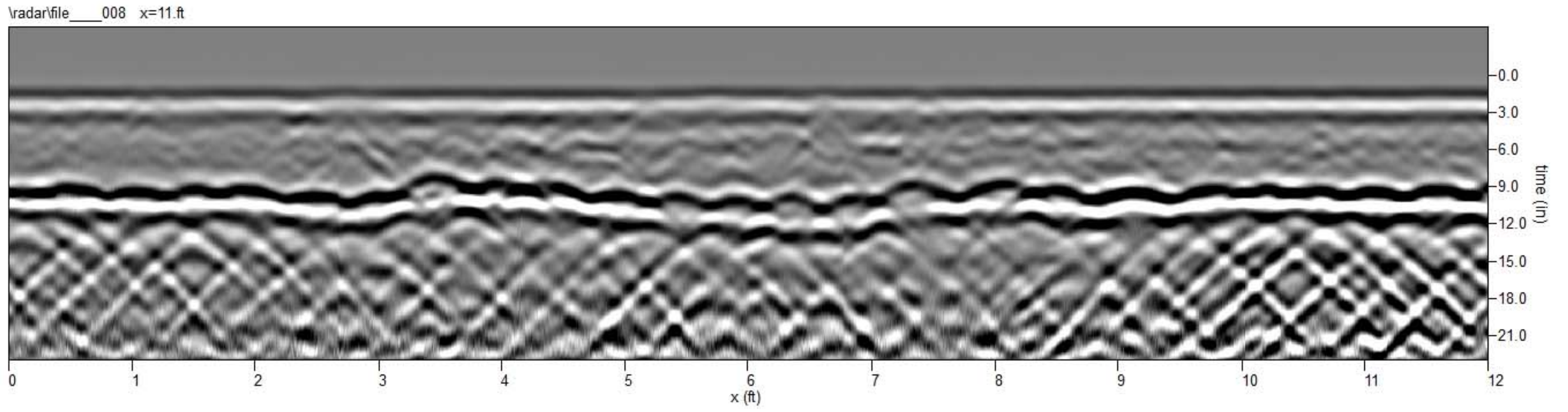
\radar\file____006   x=13.ft



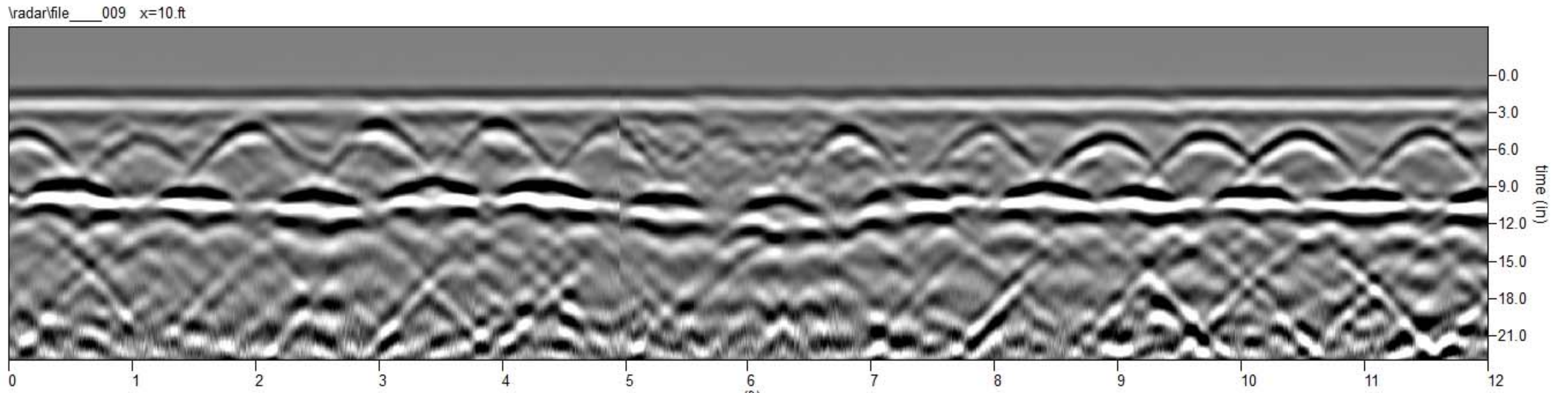
\radar\file____007   x=12.ft



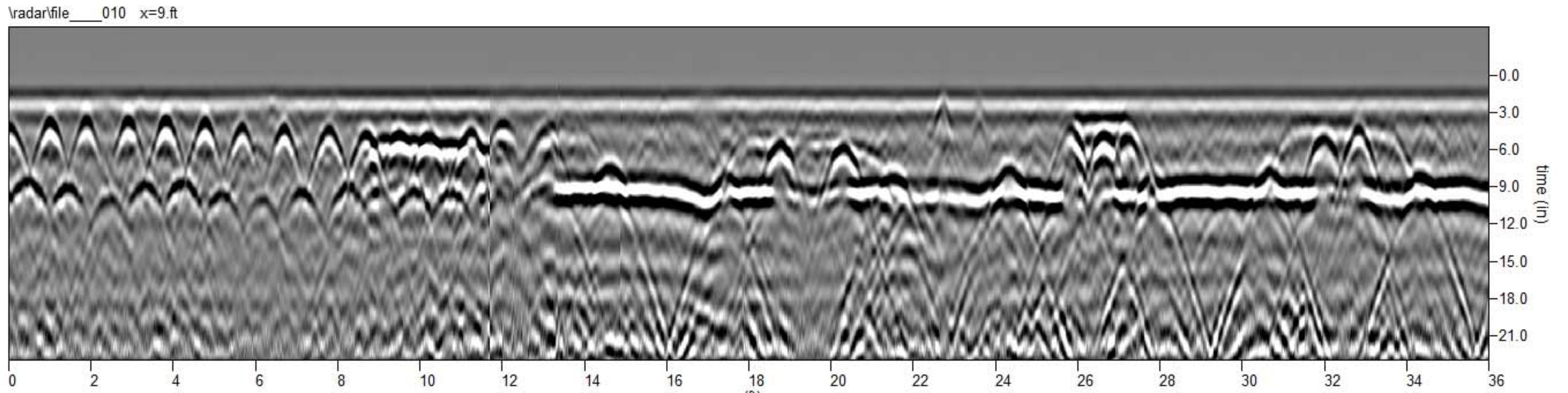
\radar\file____008   x=11.ft



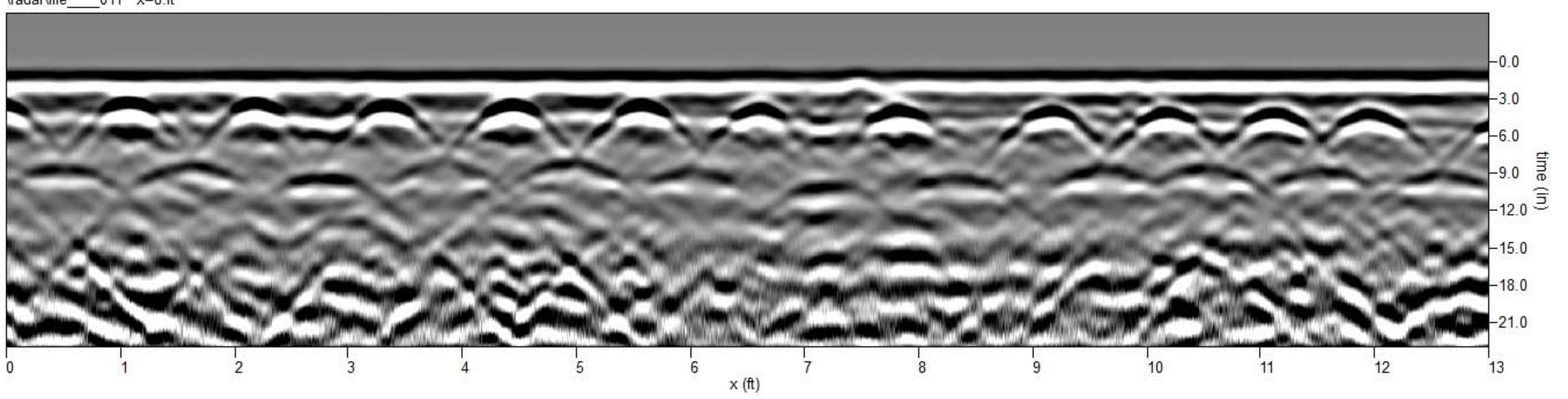
\radar\file____009   x=10.ft



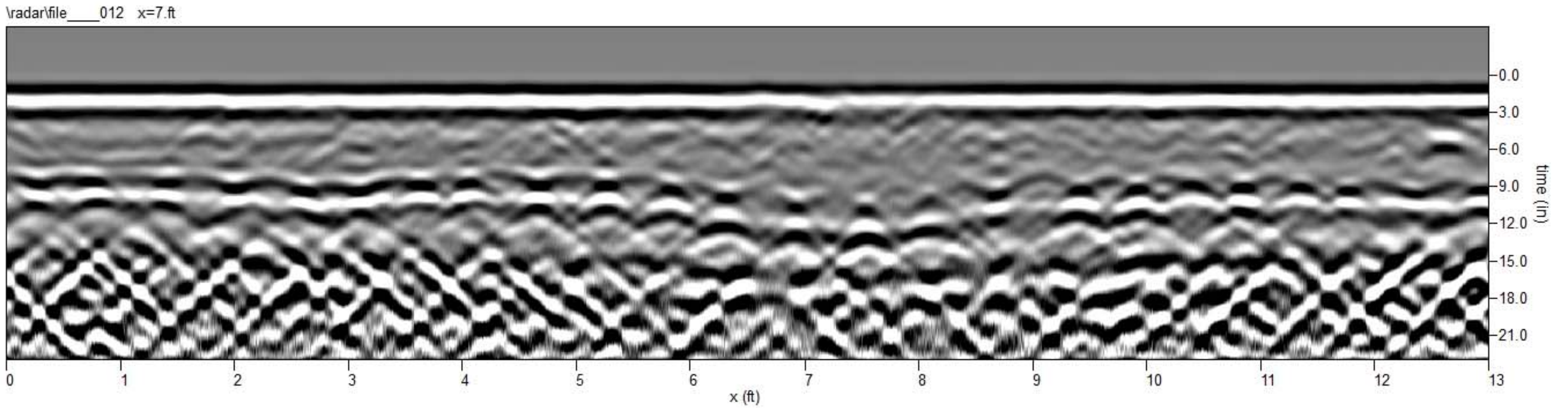
\radar\file____010   x=9.ft



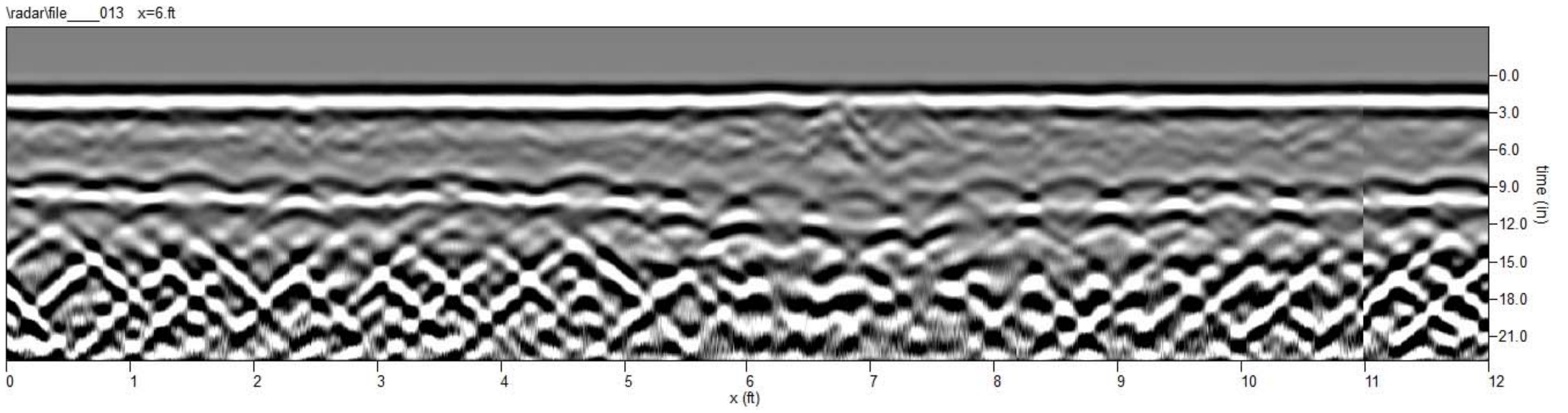
\radar\file____011   x=8.ft



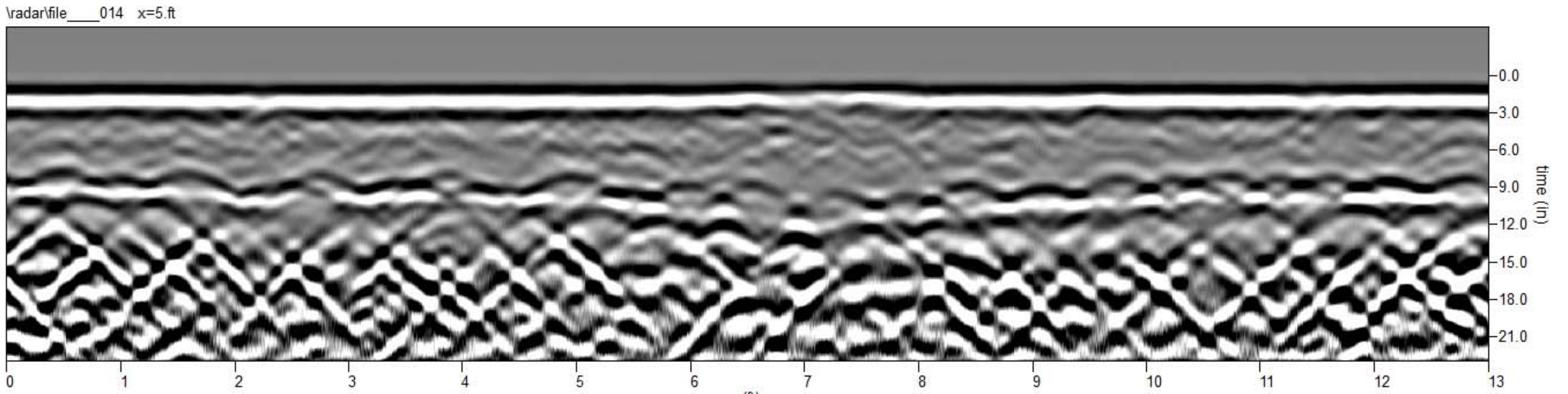
\radar\file____012   x=7.ft



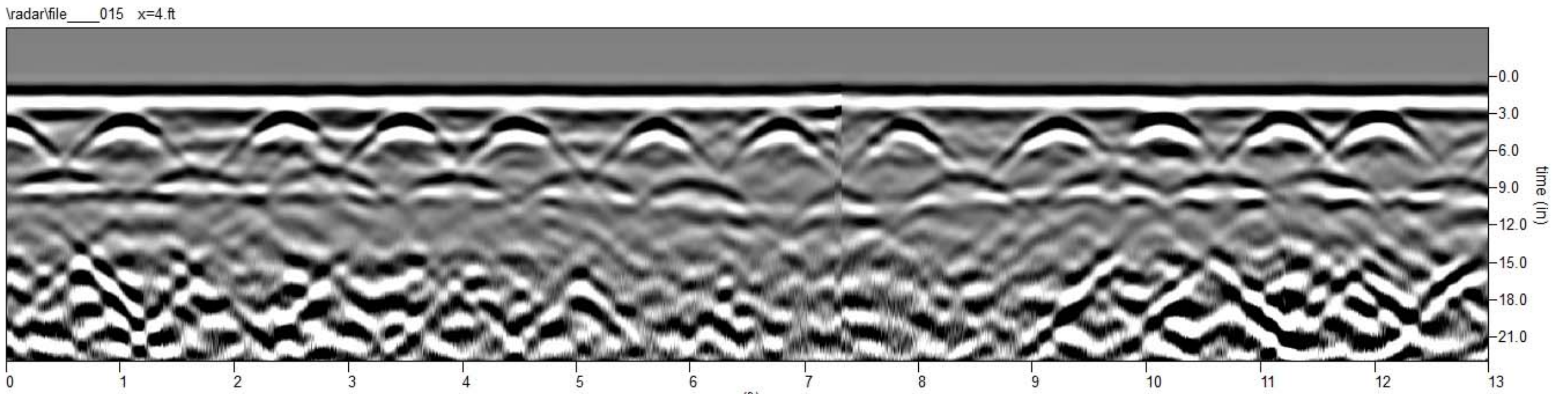
\radar\file____013   x=6.ft



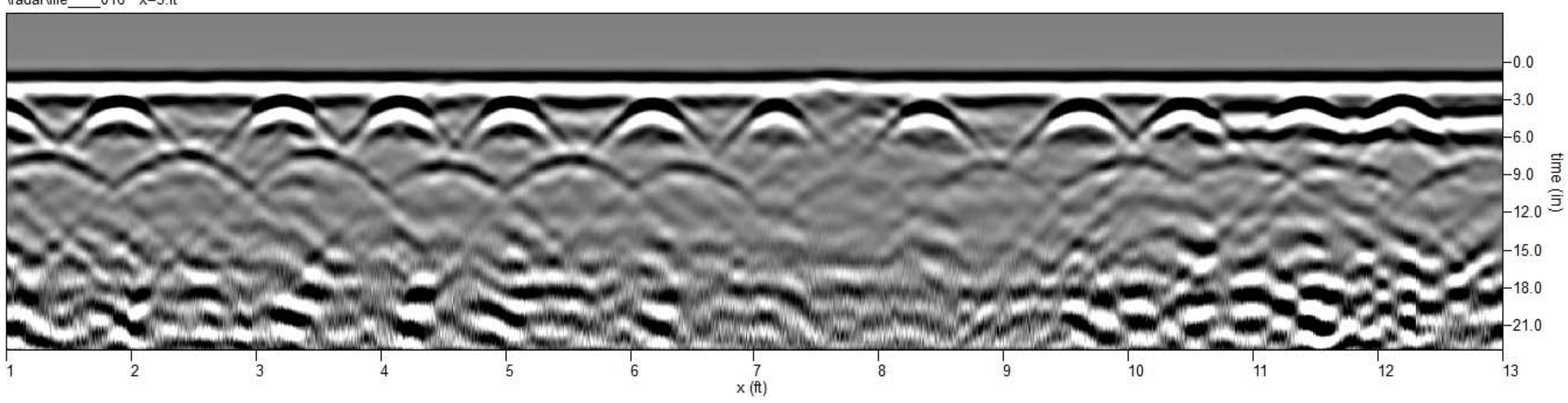
\radar\file____014   x=5.ft



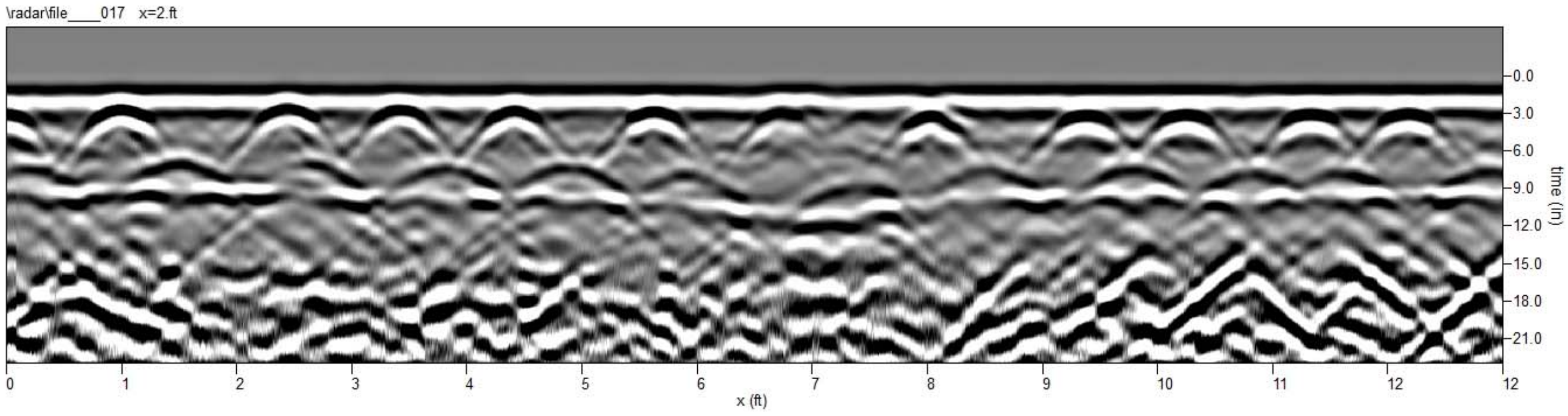
\radar\file____015   x=4.ft



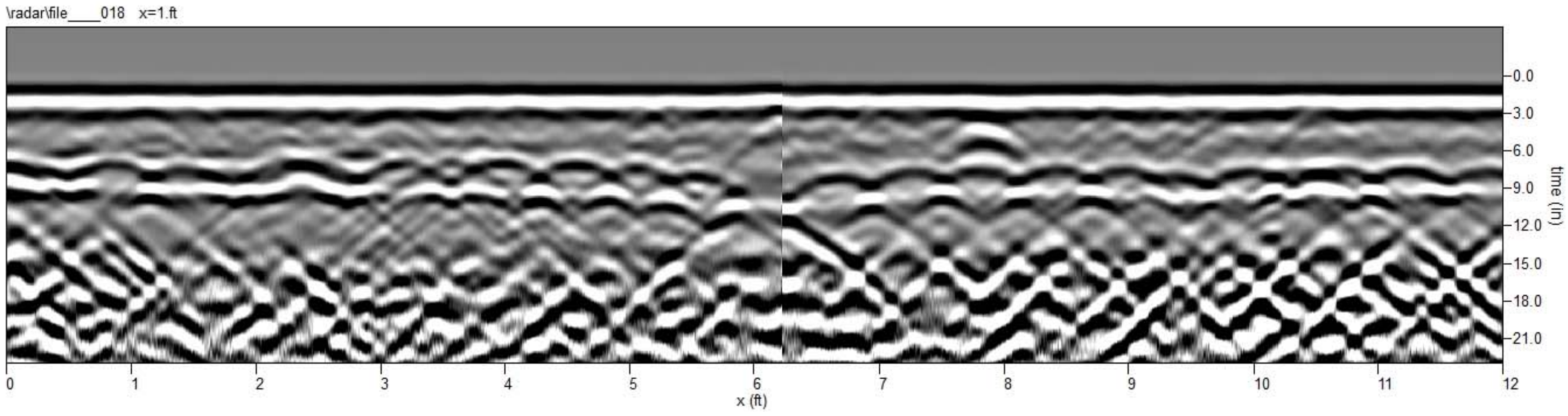
\radar\file____016   x=3.ft



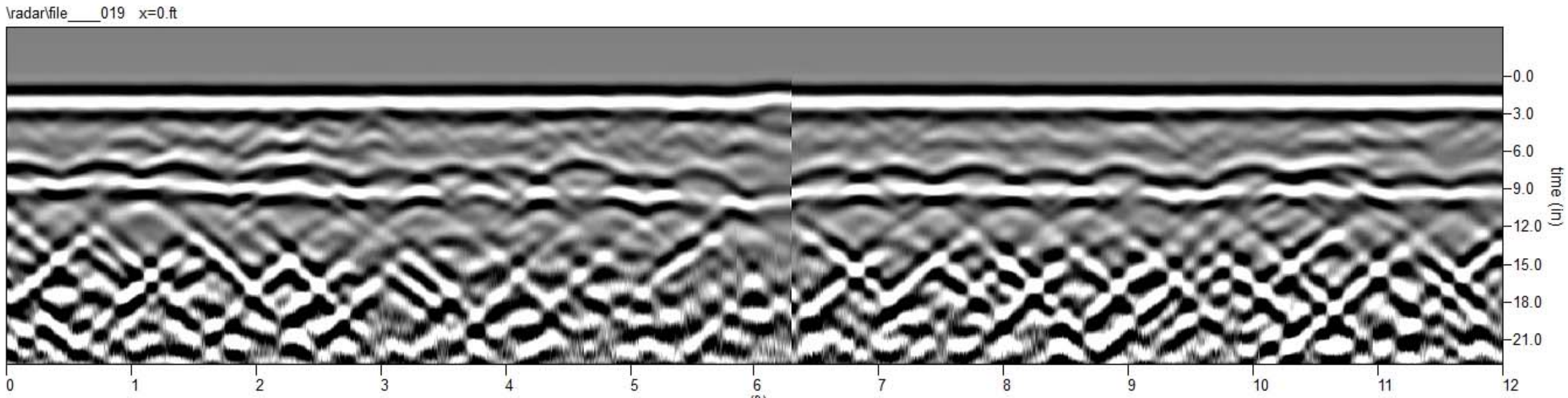
\radar\file____017   x=2.ft



\radar\file____018   x=1.ft

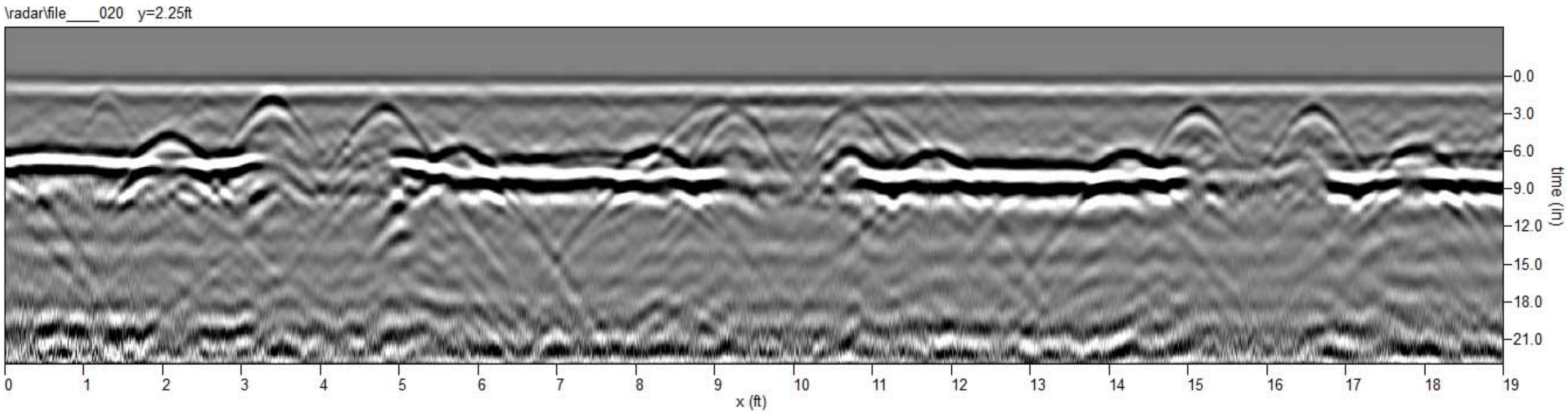


\radar\file____019   x=0.ft

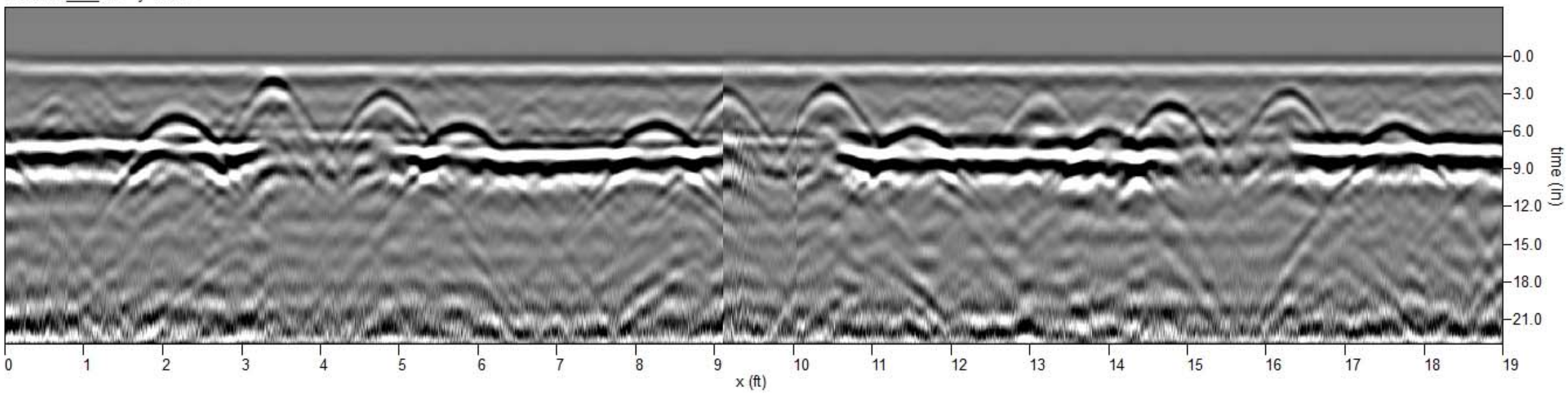


ABUTMENT 1 - TRANSVERSE SCANS

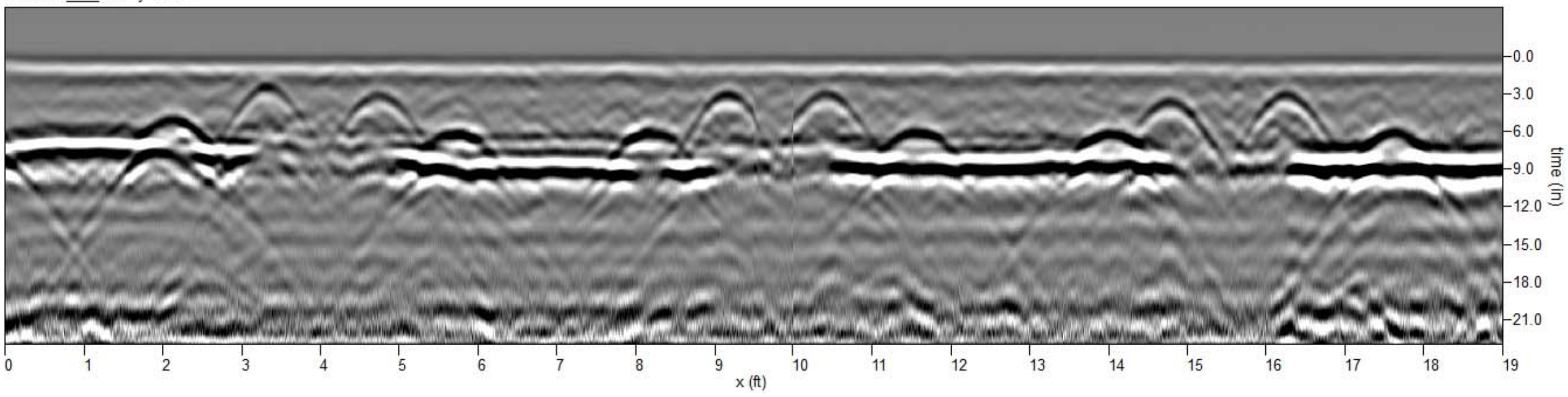
\radar\file____020 y=2.25ft



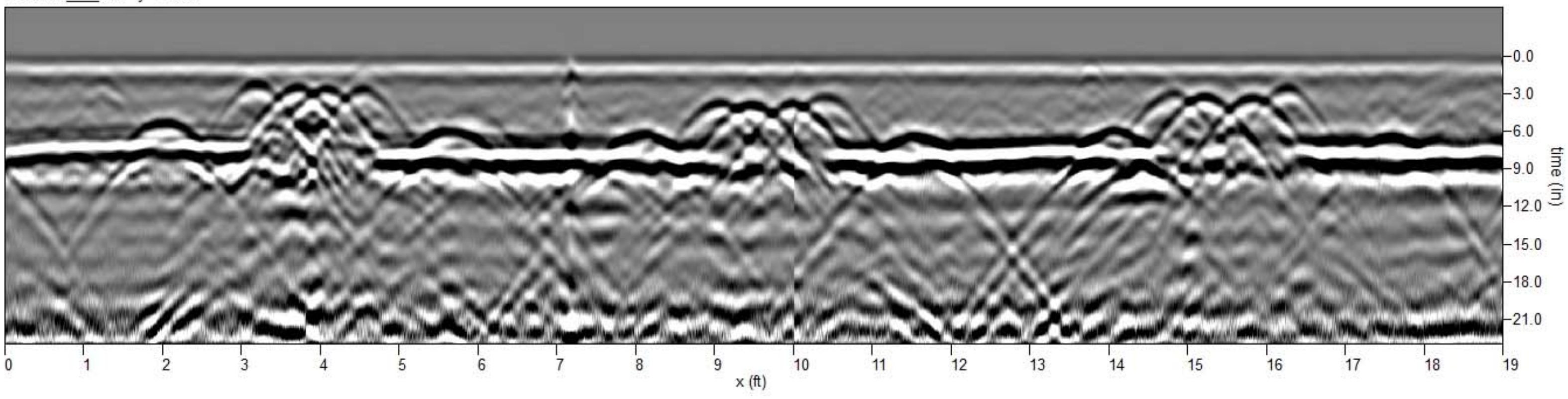
\radar\file____021 y=4.83ft



\radar\file____022 y=7.75ft

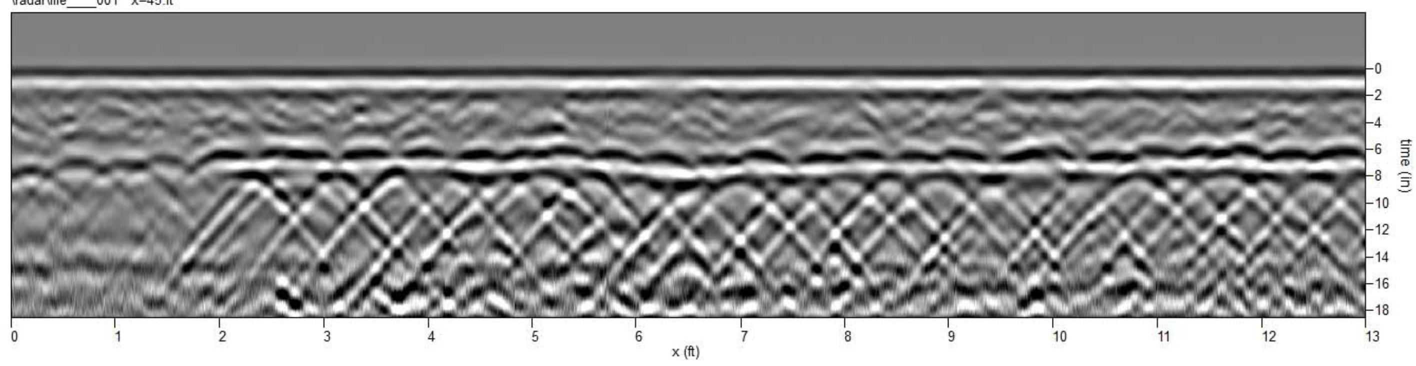


\radar\file____023 y=11.66ft

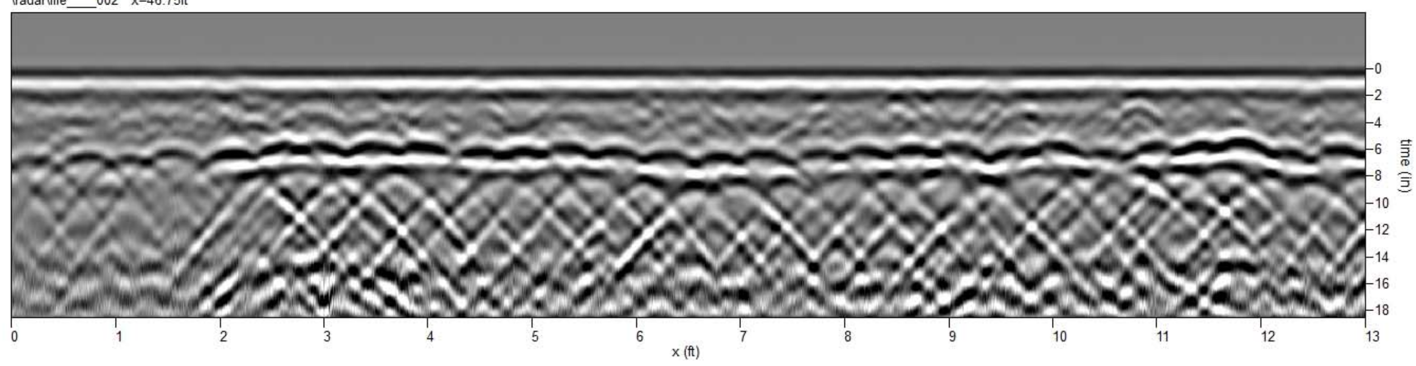


ABUTMENT 5 – LONGITUDINAL SCANS

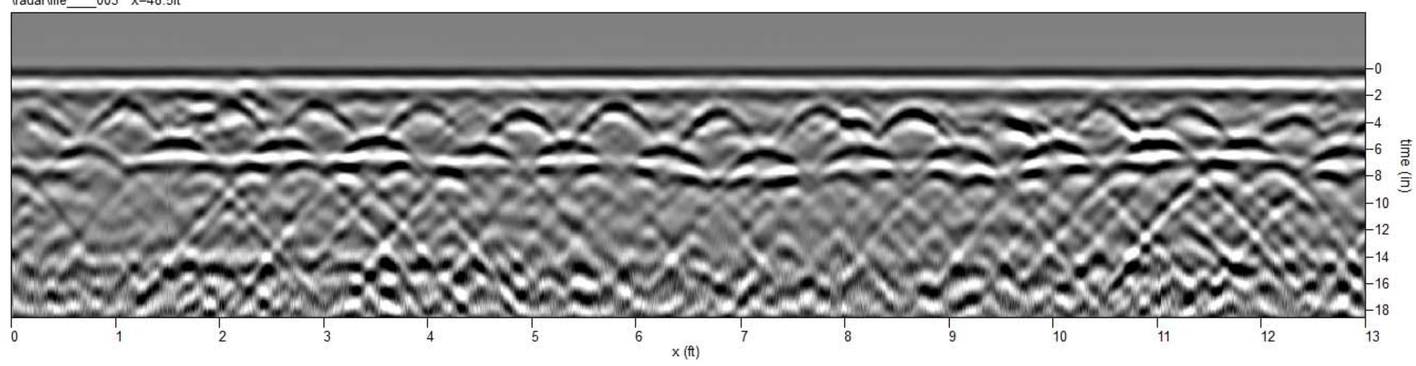
\radar\file____001   x=45.ft



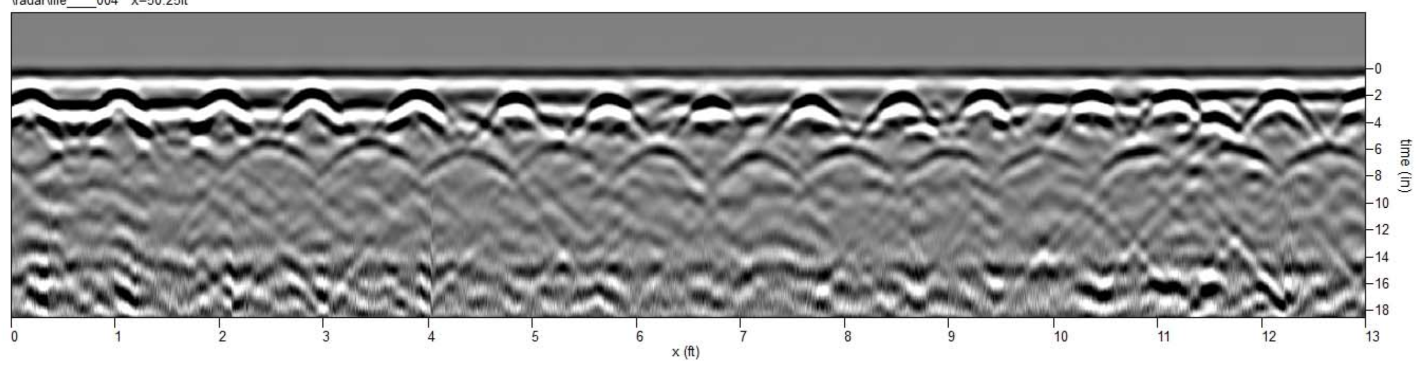
\radar\file____002   x=46.75ft



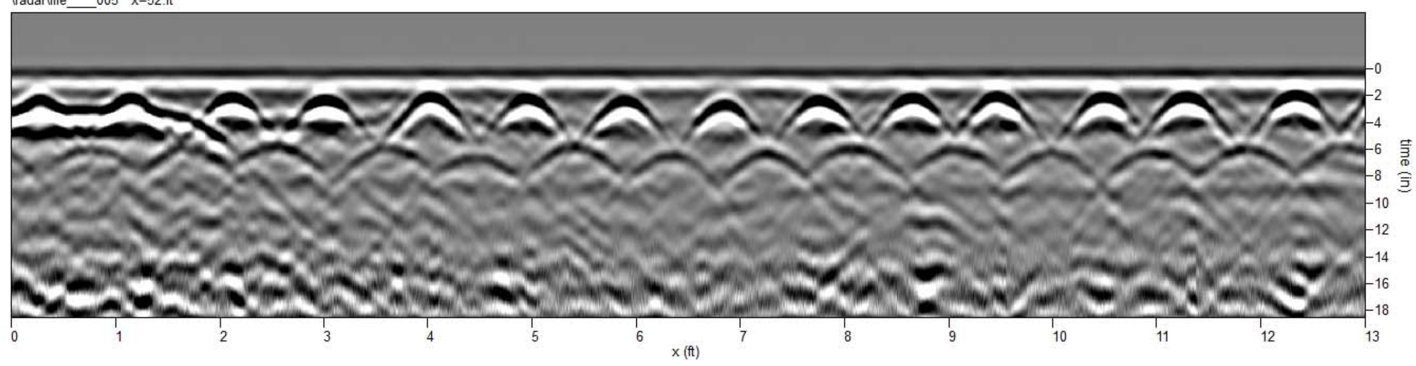
\radar\file____003   x=48.5ft



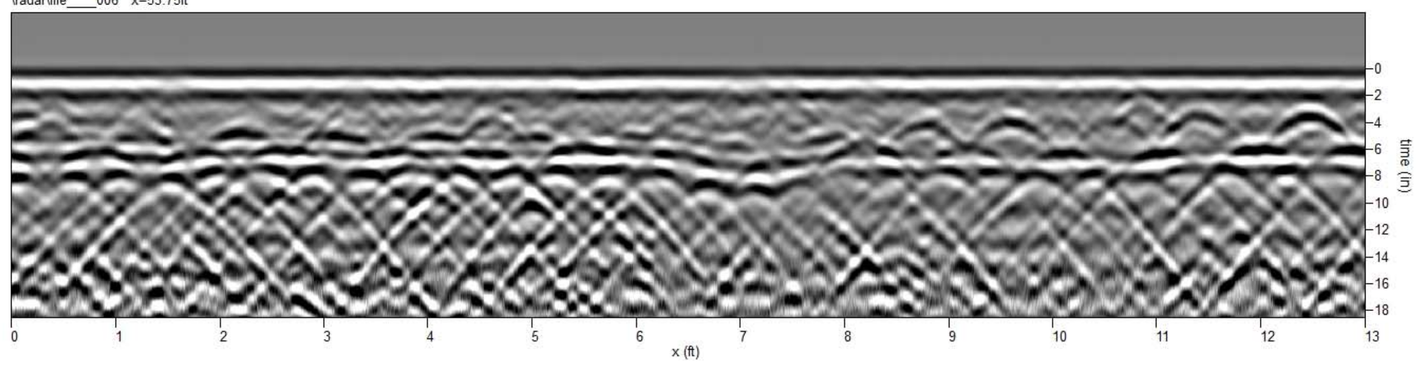
\radar\file____004   x=50.25ft



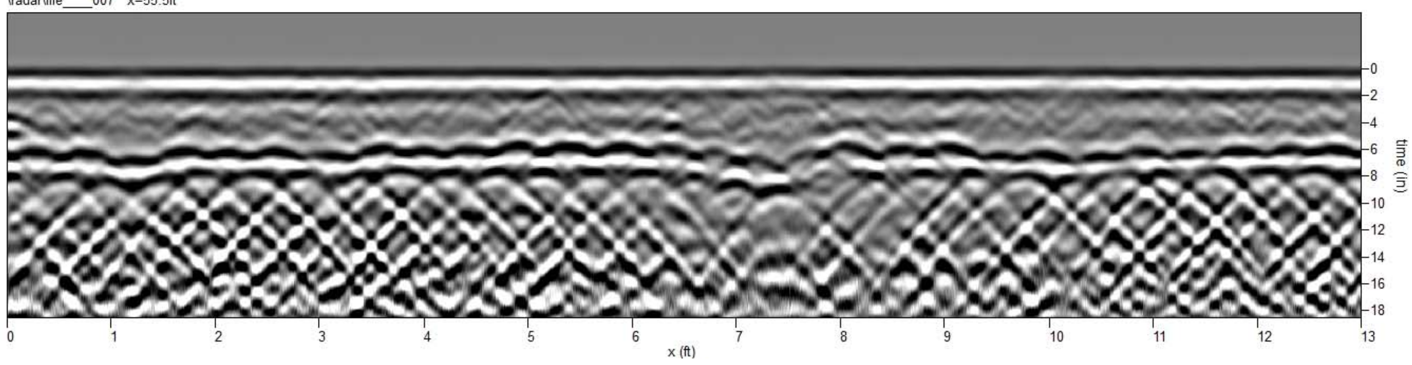
\radar\file____005   x=52.ft



\radar\file____006   x=53.75ft



\radar\file____007   x=55.5ft



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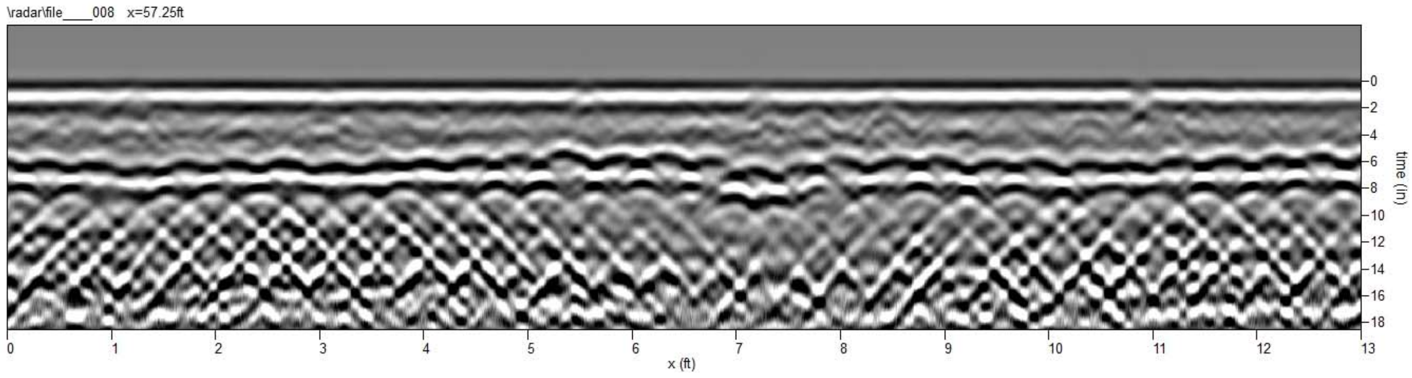

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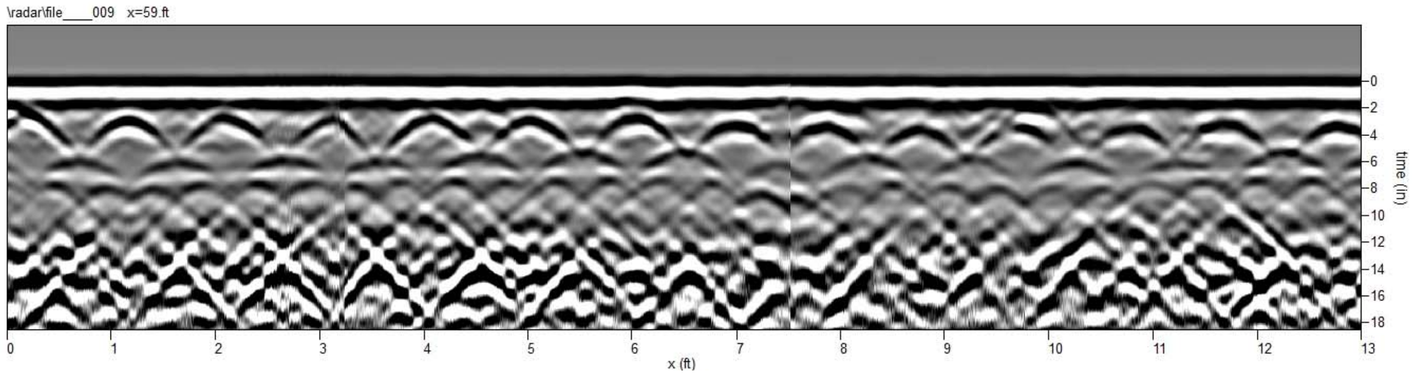
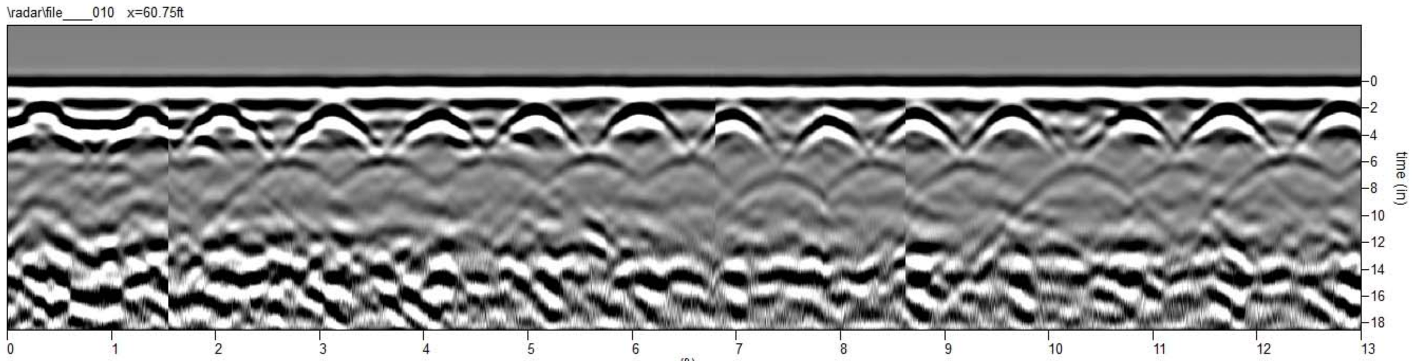
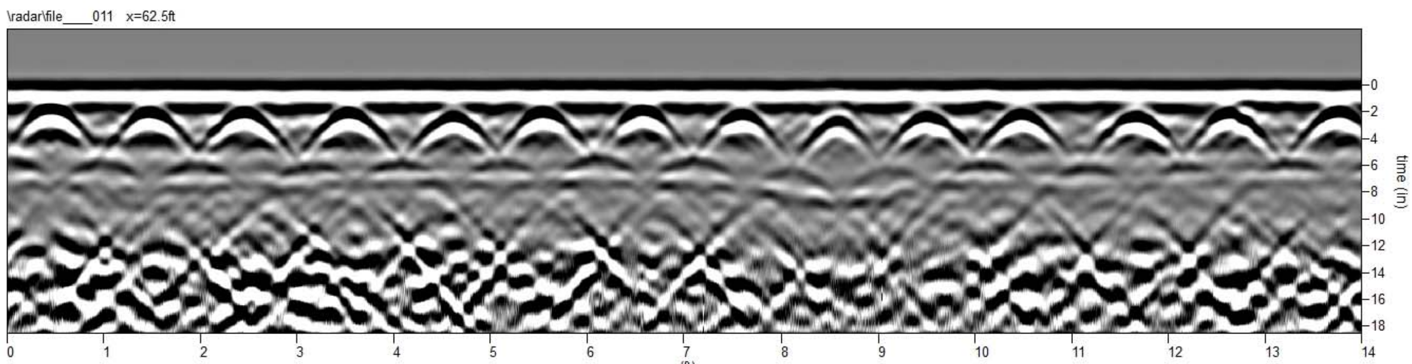


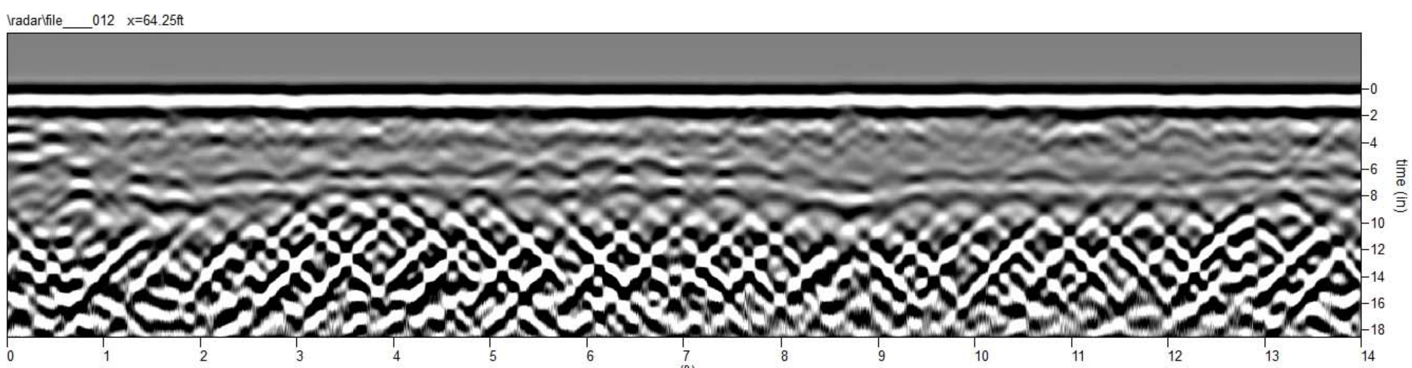
\radar\file____010   x=60.75ft



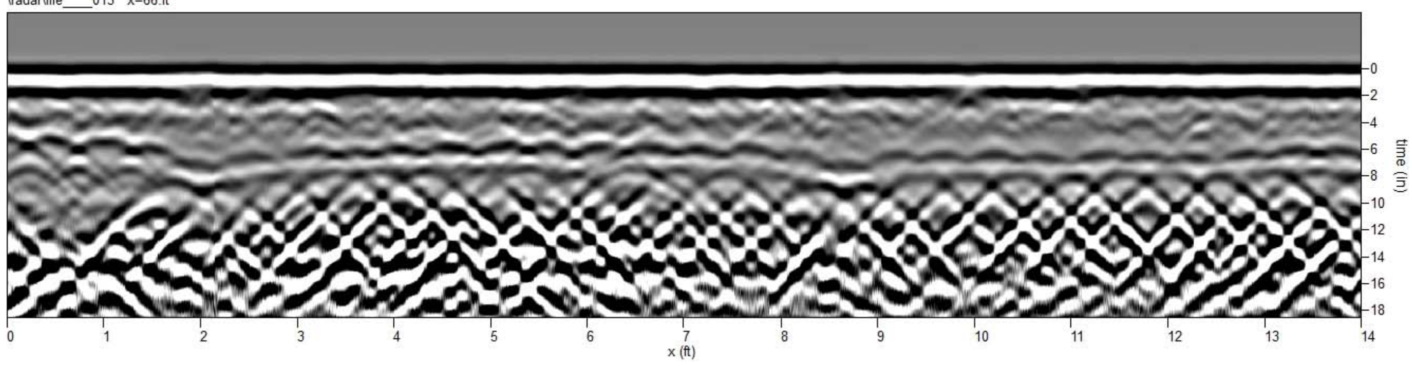
\radar\file____011   x=62.5ft



\radar\file____012   x=64.25ft



\radar\file____013 x=66.ft



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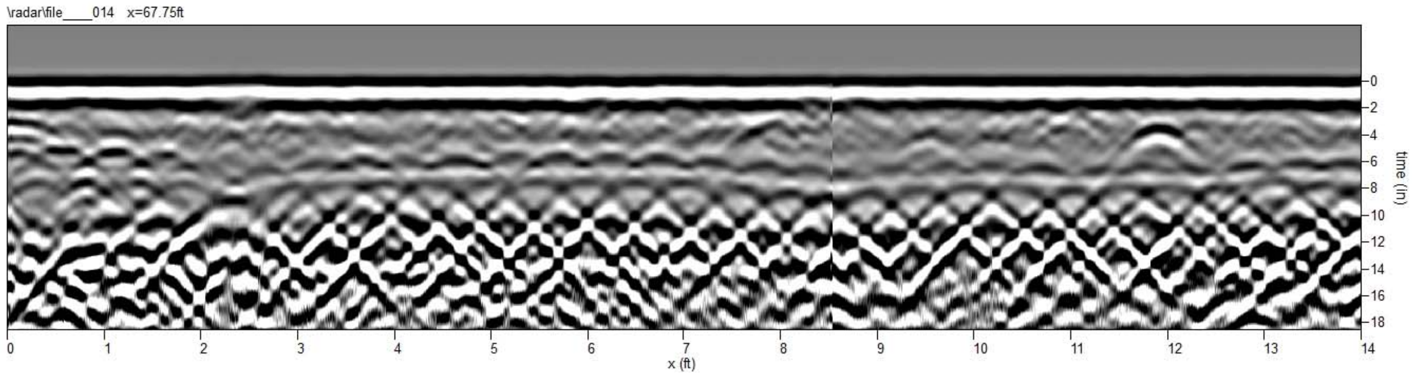

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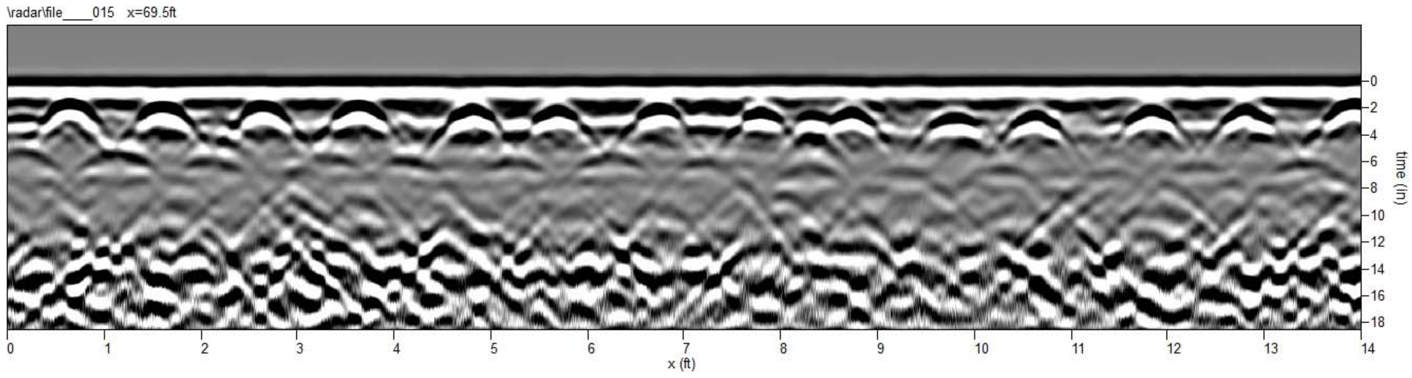

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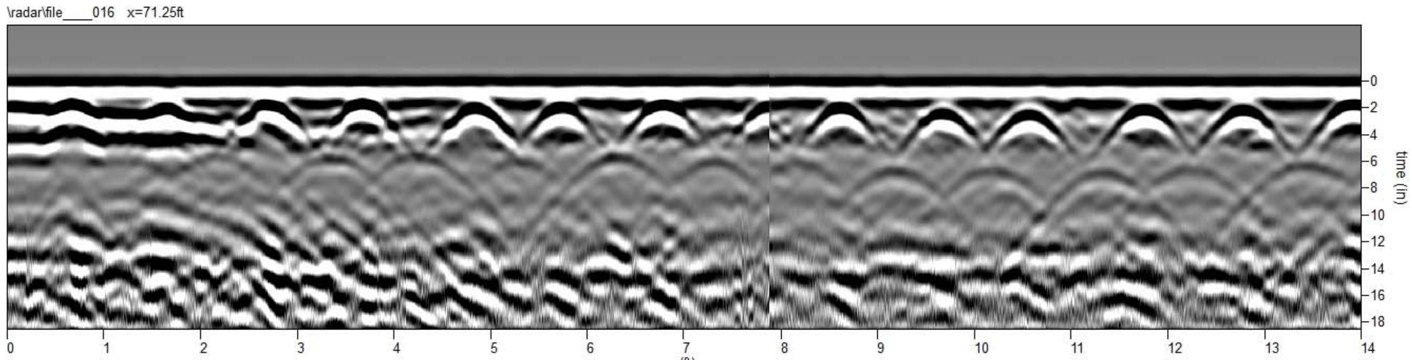

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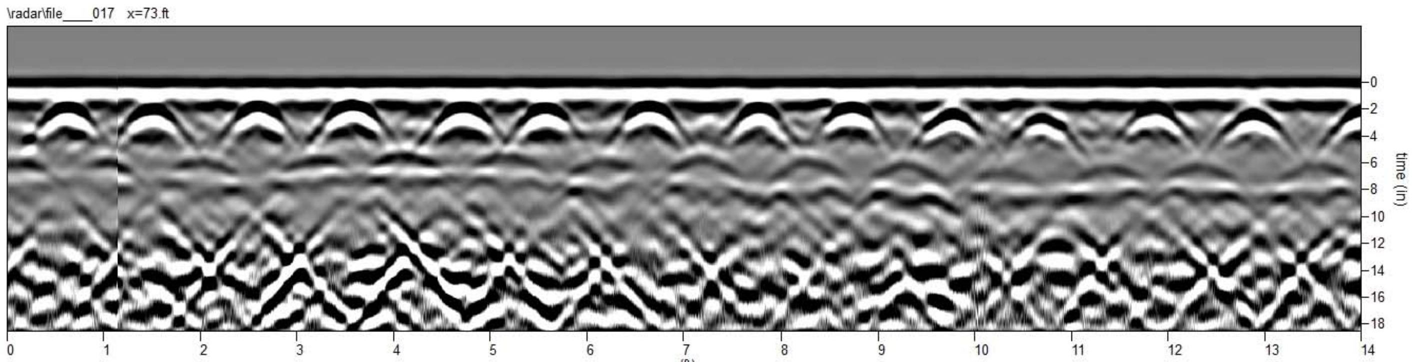

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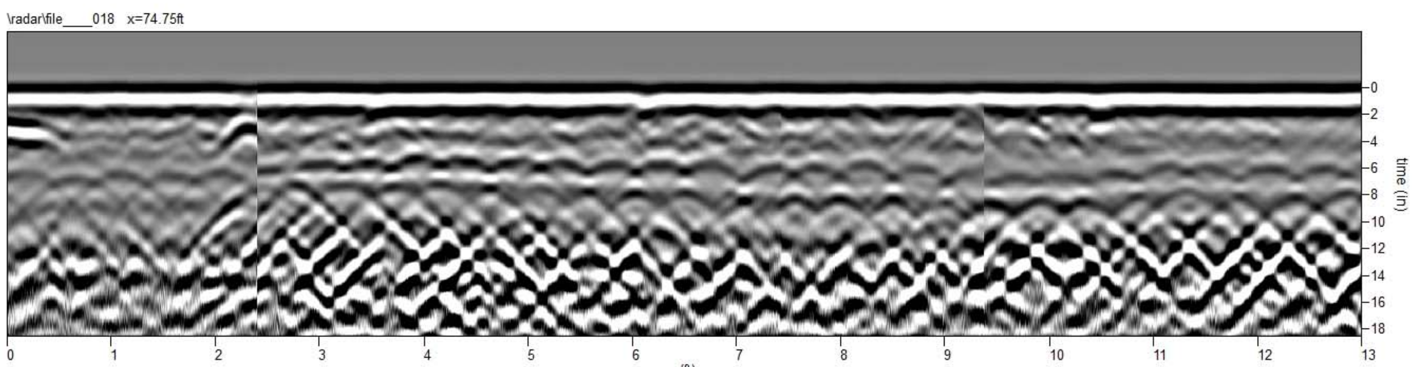

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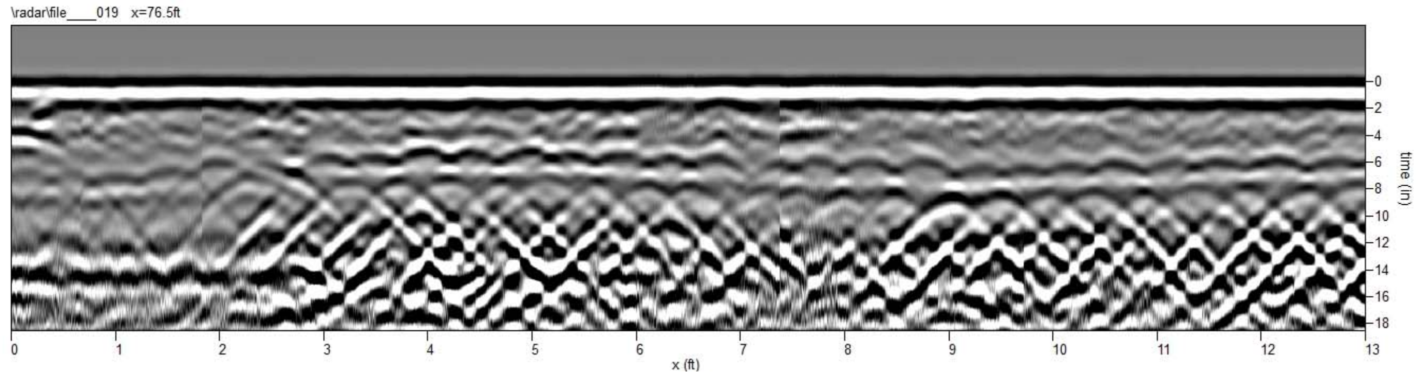
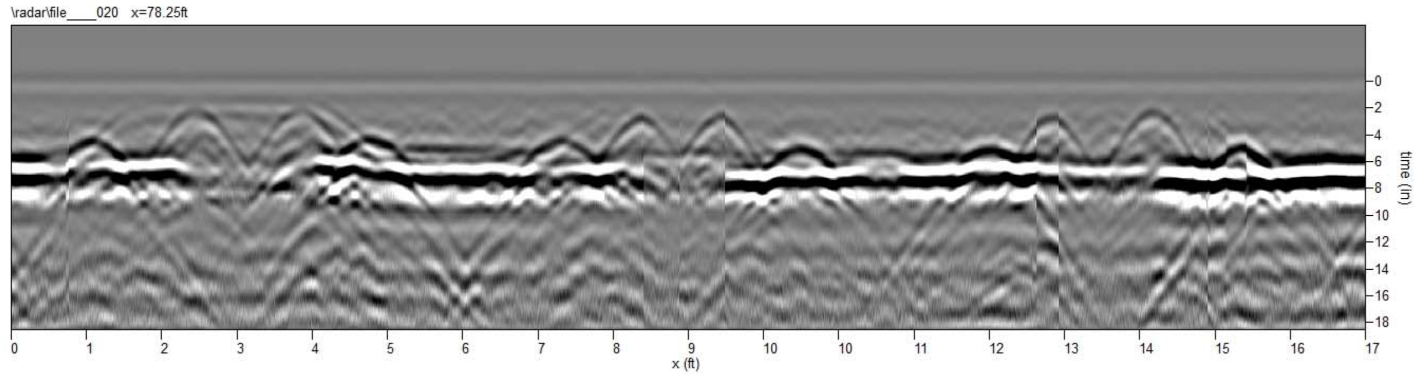
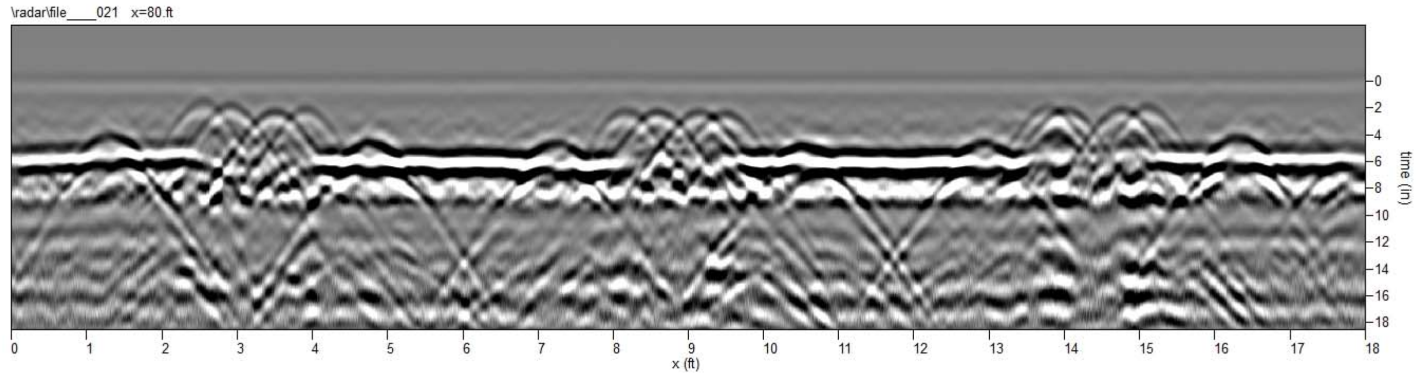


ABUTMENT 5 – TRANSVERSE SCANS

\radar\file____020   x=78.25ft



\radar\file____021   x=80.ft

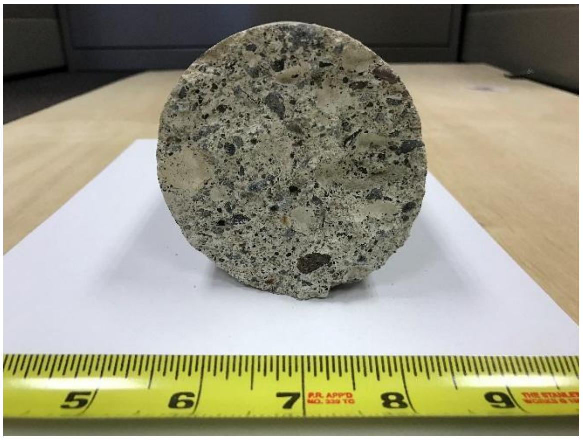


Appendix 6 – Concrete Core Images and Compressive Strength Test Data

Core 1B








Core 2






Core 3

Core 4






BSK ASSOCIATES
324 Earhart Way
Livermore, Ca 94551
Phone (925) 315-3151
Fax (925) 315-3152

Concrete Core Compression Test Report

Client:	Justin Chen	BSK Project No.:	C16-523-60L
Business Name:	Alta Vista Solutions	Sample ID No.:	16-936
Address:	3260 Blume Drive, Suite 500	Permit No.:	
City/State/Zip:	Richmond, California 94806	Report Date:	12/8/2016

Project: Stevenson Bridge
Project Address:
City, State:
Structure: Concrete Structure
Core Date:
Specified Strength, (psi):

Sample	Date Tested	Tested By	Dimensions: Average Diameter (in)	Dimensions: Average Length (in)	Area (in^2)	Ratio (L/D)	Correction Factor	Break Type	Maximum Load (lbs)	Compressive Strength (psi)
1B	12/08/16	R. Cortez	3.63	4.70	10.35	1.29	0.935	3	39,405	3,560
2	12/08/16	R. Cortez	3.63	7.48	10.35	2.06	1.000	3	29,625	2,860
3	12/08/16	R. Cortez	2.66	5.12	5.56	1.92	1.000	3	19,400	3,490
4	12/08/16	R. Cortez	2.66	5.02	5.56	1.89	1.000	2	20,400	3,670
									Average	**3,400**

Time Sampled:
Sampled by: Client
Date Delivered: 12/06/16
Delivered by: Client

TYPE 1 = CONE
TYPE 2 = CONE/SPLIT
TYPE 3 = COLUMNAR
TYPE 4 = SHEAR
TYPE 5 = SIDE FRACTURES AT TOP OR BOTTOM
TYPE 6 = SIMILAR TO TYPE 5 BUT END OF CYLINDER IS POINTED

Remarks: Nominal maximum aggregate size appears to be 2".

Cores were sampled and cured in accordance with ASTM C-42.
Cores were tested in accordance with ASTM C-39.

Signature:

Date: 12/12/16

cc:

Appendix H - As-Built Plan


SHEET NO. 1
RUMSEY BRIDGE
The entire superstructure consisting of the two arch spans will be constructed in accordance with the plans and relative detail drawings as shown on this sheet.
Revised plans and details covering the other parts of the structure, are shown on Sheet No. 2.
SOUTH ABUTMENT
LONGITUDINAL SECTION THRU RIB, HANGERS, FLOOR SLAB & GIRDERS
PLAN OF GIRDER ARRANGEMENT AND FLOOR SYSTEM
PIER NO 1
SECTION THRU COLUMNS, GIRDER AND SILL
PLAN OF FOOTING PIERS 1 AND 2
PIER NO 2
SEC- SUPERSTRUCTURE AT C-C AND SIDE AND CROSS GIRDERS AT ALL INTERSECTIONS THEREOF
HALF SECTION AT CENTER OF ARCH SPANS
HALF SECTION OF GIRDER SHORE SPANS AT CENTER OF SPAN
MICROFILMED
PLAN OF PIER NO 3
PIER NO 3
PLAN OF NORTH ABUTMENT AND FLOOR SYSTEM
REINFORCED CONCRETE BRIDGE ACROSS PUTAH CREEK FOR YOLO AND SOLANO COUNTIES
Bridge No. 95A-26.83
Bridge No. 95A-0.0
Also Rumsey Bridge ?
I-9B
~~C-17~~ 95A.11

Appendix I - Caltrans Bridge Inspection Reports

California Department of Transportation
Division of Maintenance

Structure Maintenance and Investigations

BRIDGE

INSPECTION

RECORDS

INFORMATION

SYSTEM

The requested documents have been generated by BIRIS.

These documents are the property of the California Department of Transportation and should be handled in accordance with Deputy Directive 55 and the State Administrative Manual.

Records for "Confidential" bridges may only be released outside the Department of Transportation upon execution of a confidentiality agreement.

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 03/25/2015

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: Two span RC tied arches on RC 2-column piers with RC (5) girder approach spans (Spans 1 & 4) with RC diaphragm abutments with monolithic wing walls (20 FT each). Abutments are founded on spread footings, pier columns are founded on timber piles.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

SAFE LOAD CAPACITY AND RATINGS

Design Live Load: UNKNOWN
Inventory Rating: RF=0.75 =>24.3 metric tons Calculation Method: LOAD FACTOR
Operating Rating: RF=1.26 =>40.8 metric tons Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3: Legal Type 3S2: Legal Type 3-3: Legal

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m Net Width: 6.1 m No. of Lanes: 2 Speed: 55 mph
Min. Vertical Clearance: 4.31 m Overlay Thickness: 0.0 Inches
Rail Code: 0000

Rail Type	Location	Length (ft)	Rail Modifications
Misc. Concrete	Right/Left	600	

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

NOTICE

The bridge inspection condition assessment used for this inspection is based on the American Association of State Highway and Transportation Officials (AASHTO) Bridge Element Inspection Manual 2013 as defined in Moving Ahead for Progress in the 21st Century (MAP-21) federal law. The new element inspection methodology may result in changes to related condition and appraisal ratings on the bridge without significant physical changes at the bridge.

The element condition information contained in this report represents the current condition of the bridge based on the most recent routine and special inspections. Some of the notes presented below may be from an inspection that occurred prior to the date noted in this report. Refer to the Scope and Access section of this inspection report for a description of which portions of the bridge were inspected on this date.

INSPECTION COMMENTARY

SCOPE AND ACCESS
Water was flowing under Span 2 during this inspection. All elements were accessible and inspected.

SAFE LOAD CAPACITY
A Load Rating Summary Sheet dated 2/11/2010 is on file for this structure and is based on

INSPECTION COMMENTARY

hand calculations. While this inspection does not include a check of that analysis, it does verify that the structural conditions observed during this inspection are consistent with those assumed in that analysis.

OPERATIONAL SIGNS
There are signs in place at both approaches that read "NARROW BRIDGE".

WATERWAY
The BIR dated 5/9/2008 determined that this structure is Scour Critical (NBI Item 113 code of 3). A Scour plan of action dated 11/21/2008 has been completed. The Scour Plan of Action states that the channel has remained relatively stable since 1971. However, County forces will monitor this bridge when the flow rate exceeds 4,500 cfs or about 10 feet above the pile as well as an annual inspection to check for degradation and undermining. On this date a channel cross section was taken and compared to the previous stream section dated 3/29/2007. The results of that comparison indicated that the channel has degraded 8 inches at Pier 3 and 10 inches at Pier 4.

ELEMENT INSPECTION RATINGS AND COMMENTARY

Elem No.	Defect /Prot	Defect	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4
12			**Deck-RC**	2	670	sq.m	590	0	80	0
	1080		**Delamination/Spall/Patched Area**	2	52		0	0	52	0
	1130		**Cracking (RC and Other)**	2	28		0	0	28	0
	521		**Concrete Coat.(Meth/Paint/Seal)**	2	554	sq.m	554	0	0	0

(12-1080)
There are numerous soffit spalls on either side of the structure of all spans. The spalls are typically between one to two feet in length and approximately six inches wide with exposed corroding rebar. It is recommended that the exposed rebar be cleaned and painted to prevent further deterioration of the steel. Based on the photos in the last October 2009 report, the condition of the superstructure has not changed.

The following is a list of the soffit spall locations:
Span 1:
Bay 1, left side of bridge
Bay 4, right side of bridge

Span 2:
Bays 8, 13 and 15; left side of the bridge
Bay 5, 7, 8, 10 and 15; right side of the bridge

Span 3:
Bays 1, 2, 3, 4, 9, 10, and 13; left side of the bridge
Bays 6, 9, 10, 11, 12, 13 and 14; right side of the bridge

Span 4:
Bay 1, left isde of bridge
Bay 4, right side of bridge

(12-1130)
There are two transverse soffit cracks in Span 1 near Bent 2. There are also 2 transverse soffit cracks in Span 4 near Bent 4. Based on the photos included with the October 2009 BIR, the soffit cracks have not changed.

(12-521)

ELEMENT INSPECTION RATINGS AND COMMENTARY

Elem No.	Defect	Defect /Prot	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4

The large transverse deck cracks in Spans 1 & 4 along with the entire deck were recently treated with methacrylate (photos 1 & 2).

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
110		Girder/Beam-RC	2	122	m	40	54	28	0
	1080	Delamination/Spall/Patched Area	2	50		0	30	20	0
	1130	Cracking (RC and Other)	2	32		0	24	8	0

(110-1080)
There are spalls with exposed rebar on both girders. Approximately 40% of the girders have spalls or delaminations.

(110-1130)
There are cracks in all four giders in Spans 1 and 4 that progress to the soffit at these locations. The cracks are greater that 0.05 inch wide. There are cracks on the girders is Spans 2 & 3 that are estimated at 20% of the length of the girders and up to 0.05 inch wide. Based on the photos included with the October 2009 BIR, this condition has not changed.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
144		Arch-RC	2	132	m	66	33	33	0
	1080	Delamination/Spall/Patched Area	2	66		0	33	33	0

(144-1080)
There are spalls throughout the arch members which have been documented as far back as the 1993 report (photos 8 & 9, Oct. 2009 BIR). The largest spall in located in Span 2 on the southern arch at the fifth column on the left (photo 3, March 2013 BIR). Approximately 50% of the arch has spalls or delaminations.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
155		Floor Beam-RC	2	180	m	179	0	1	0
	1080	Delamination/Spall/Patched Area	2	1		0	0	1	0

(155-1080)
There is a spall with exposed rebar on the right side of Floor beam 14 in Span 3 (photo 15, Oct. 2009 BIR).

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
205		Column-RC	2	6	each	6	0	0	0

(205)
There were no significant defects noted.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
215		Abutment-RC	2	40	m	40	0	0	0

(215)
There were no significant defects noted.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
220		Pile Cap/Footing-RC	2	1	m	1	0	0	0

(220)
There were no significant defects noted. There is 58 inches of vertical exposure of the pile cap at Bent 3. During this inspection no undermining was observed. No corrective action is required at this time.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
228		Pile-Timber	2	1	ea.	1	0	0	0

(228)
The pile element is included to indicate the presence of piles on this structure. The piles were not exposed for visual inspection. No indication of pile distress was noted in any substructure element.

Elem No.	Defect	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4
331		Railing-RC	2	183	m	93	45	45	0
	1080	Delamination/Spall/Patched Area	2	30		0	0	30	0

ELEMENT INSPECTION RATINGS AND COMMENTARY

Elem No.	Defect /Prot	Defect	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4
	1130		**Cracking (RC and Other)**	2	60		0	45	15	0

(331-1080)
There are numerous random spalls and incipient spall on both bridge rails. Many of the spalls have been patched; however, the patches are beginning to fail (break up and spall). Eleven rail posts along a section of the left rail in Span 1 have been hit by traffic or have severely deteriorated (photo 1, March 2013 BIR).

(331-1130)
There are numerous random cracks on both bridge rails. The most severe is a three inch wide crack/spall on the left rail over Bent 3 at the connection to the northern arch (photo 2, March 2013 BIR).

WORK RECOMMENDATIONS

RecDate: 10/23/2009
Action : Super-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:

Clean and paint the exposed rebar to prevent further deterioration.

CHANNEL X-SECTION

Side : Upstream
Measured From :Top of concrete rail

X-Section Date: 03/25/2015

Location	Horiz(m)	Vert(m)	Comments
Abutment 1	1.00	5.40	
Pier 2	12.00	11.80	
	26.00	15.05	edge of water
	32.00	16.90	Thalweg
	35.00	15.70	Edge of water
Pier 3	45.00	15.70	
Pier 4	57.00	7.35	

Team Leader : W. John Baker

Report Author : W. John Baker

Inspected By : W.Baker/RC.Dills

6/24/15

W. John Baker (Registered Civil Engineer) (Date)

REGISTERED PROFESSIONAL ENGINEER
W. John Baker
No. 60307
06/30/2016
CIVIL
STATE OF CALIFORNIA

Printed on: Wednesday 06/24/2015 10:05 AM

23C0092/AAAK/31719

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **23C0092**
(5) INVENTORY ROUTE(ON/UNDER)- ON 140000000
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED- PUTAH CREEK
(7) FACILITY CARRIED- STEVENSON BR RD
(9) LOCATION- SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK- NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 11.31 SEC
(17) LONGITUDE 121 DEG 51 MIN 03.92 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- ARCH - THRU CODE 212
(44) STRUCTURE TYPE APPR:MATERIAL- CONCRETE CONT
TYPE- TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- NONE CODE 0
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 789
(30) YEAR OF ADT 2008 (109) TRUCK ADT 5 %
(19) BYPASS, DETOUR LENGTH 19 KM

*************** GEOMETRIC DATA ****************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0.2 M RIGHT 0.2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 60.4
STATUS FUNCTIONALLY OBSOLETE
HEALTH INDEX 86.9
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- NOT ON NHS 0
(26) FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY- NOT STRAHNET 0
(101) PARALLEL STRUCTURE- NONE EXISTS N
(102) DIRECTION OF TRAFFIC- 2 WAY 2
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- COUNTY HIGHWAY AGENCY 02
(22) OWNER- COUNTY HIGHWAY AGENCY 02
(37) HISTORICAL SIGNIFICANCE- ELIGIBLE 2

**************** CONDITION **************** CODE

(58) DECK 7
(59) SUPERSTRUCTURE 7
(60) SUBSTRUCTURE 8
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- UNKNOWN 0
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 40.8
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 24.3
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 6
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0000
(113) SCOUR CRITICAL BRIDGES 3

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $1,541,000
(95) ROADWAY IMPROVEMENT COST $308,200
(96) TOTAL PROJECT COST $2,588,880
(97) YEAR OF IMPROVEMENT COST ESTIMATE 2010
(114) FUTURE ADT 1518
(115) YEAR OF FUTURE ADT 2035

*************** INSPECTIONS ***************

(90) INSPECTION DATE 03/15 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

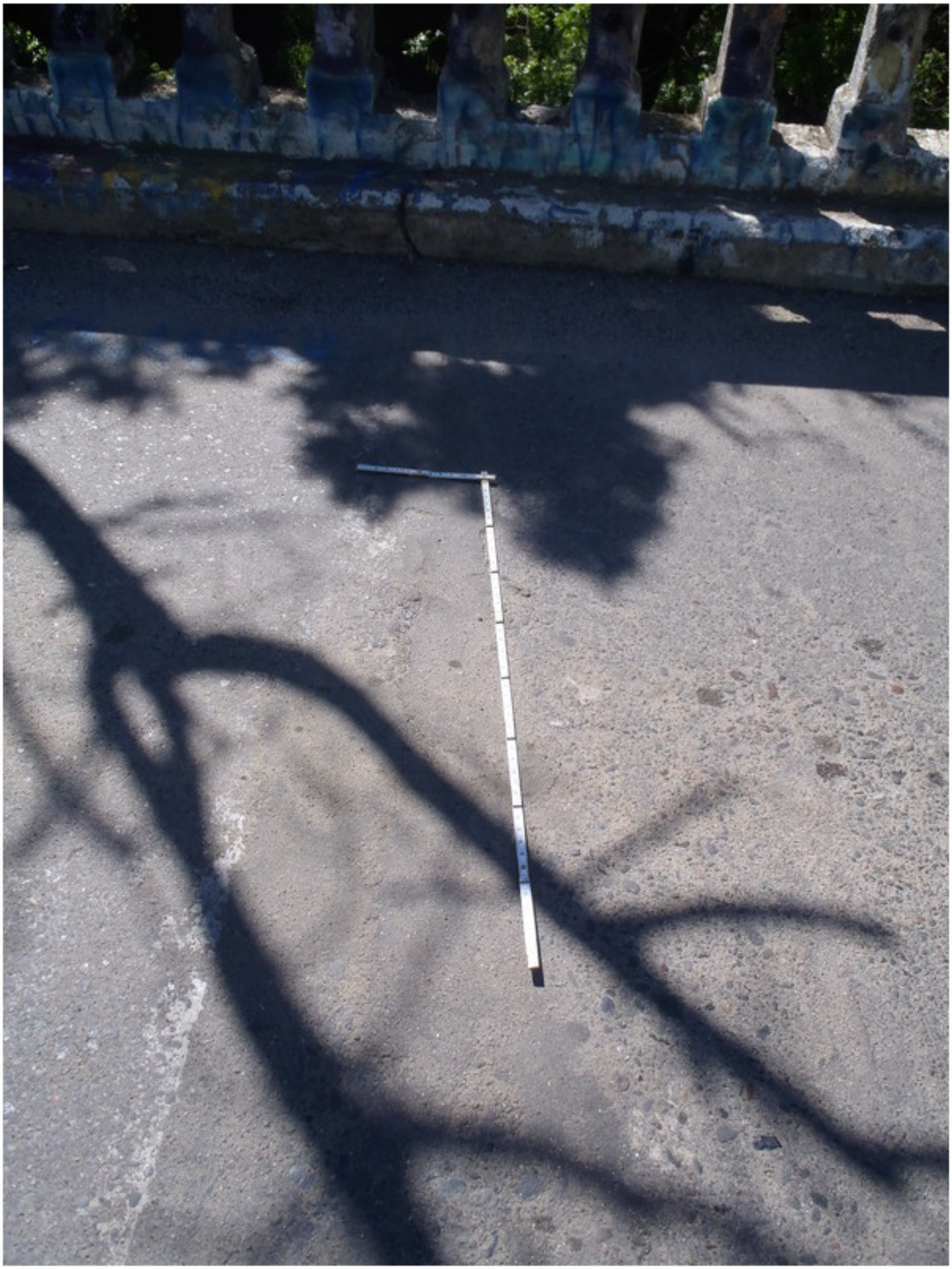


Photo No. 1
Transverse crack in Span 1 and remainder of deck are treated with methacrylate.

Photo No. 2
Transverse crack in Span 4 and remainder of deck are treated with methacrylate.

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 03/28/2013

Inspection Type

Routine	FC	Underwater	Special	Other
X				

Bridge Inspection Report

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers which are founded on timber piles. RC, seat type abutments which are founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

SAFE LOAD CAPACITY AND RATINGS

Design Live Load: UNKNOWN
Inventory Rating: RF=0.75 =>24.3 metric tons — Calculation Method: LOAD FACTOR
Operating Rating: RF=1.26 =>40.8 metric tons — Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3: Legal — Type 3S2: Legal — Type 3-3: Legal

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m — Net Width: 6.1 m — No. of Lanes: 2 — Speed: 55 mph
Min. Vertical Clearance: 4.31 m

Rail Code: 00N0

Rail Type	Location	Length (ft)	Rail Modifications
Misc. Concrete	Right/Left	600	

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

INSPECTION COMMENTARY

SCOPE AND ACCESS
Water was flowing under Span 2 during this inspection. All elements were accessible and inspected.

REVISIONS
NBI Item No. 36c, "Traffic Safety Features", was modified from "1" to "N", there is no approach guard rail.

DECK AND ROADWAY:
There are numerous random cracks, spalls and incipient spall on both bridge rails. Many of the spalls have been patched; however, the patches are beginning to fail (break up and spall). Eleven (11) rail posts along a section of the left rail in Span 1 have been hit by traffic or have severely deteriorated (photo 1). There is a three inch wide crack/spall on the left rail over Bent 3 at the connection to the northern arch (photo 2). These spalls are not structurally significant and require no corrective action.

There is a full width transverse deck crack in Span 1 near Bent 2 which was first documented in the 2007 report. This cracks measures 0.4 inch wide at the top of the right exterior girder and 0.6 inch wide at the top of the left exterior girder. There is also a

INSPECTION COMMENTARY

similar transverse deck crack in Span 4 near Bent 4 which is not as severe as the Span 1 crack. This cracking appears to have been caused by settlement of the abutments which are founded on spread footings.

The deck has transverse cracks spaced between four to eight (4'-8') feet on center with edge spalling measuring up to 1/4 inch wide. These cracks appear to correspond with the locations of the floor beams. This condition was first documented during the 1990 inspection.

Based on the photos in the October 2009 report, the condition of the deck and rail have remained much the same.

Currently there is an outstanding work recommendation to treat the deck with Methacrylate that is still valid. A call was made and an email was sent on July 2, 2013 to the Engineering Manager at Solano County Public works, Matt Tuggle (707-784-6072), in an attempt to determine if any work is scheduled for this structure. As of July 17, 2013, the county has not responded to this request for information.

SUPERSTRUCTURE:
There are cracks and spalls in the arch members which have been documented as far back as the 1993 report. The largest spall in located in Span 2 on the southern arch at the fifth column on the left (photo 3). These conditions were caused by vehicle impacts as well as a lack of cover over the rebar. At this time the current severity of this condition does not require analysis or corrective action.

There are two (2) transverse soffit cracks in Span 1 near Bent 2. There are also 2 transverse soffit cracks in Span 4 near Bent 4. These cracks are reflective of the cracking noted in the deck at these locations. In the areas surrounding these cracks there is water staining as wells as locations of efflorescence and brown staining. This provides visual evidence that water is seeping through the slab.

There are spalls in the girders with exposed corroding rebar.

Numerous soffit spalls are on either side of the structure of all spans. The spalls are typically between one to two (1'-2') feet in length and are six (6") inches wide with exposed corroding rebar.

The spalling conditions noted above appear to have been caused by a lack of cover over the steel during construction. It is recommended that the exposed rebar be cleaned and painted to prevent further deterioration of the steel. Based on the photos in the last October 2009 report, the condition of the superstructure has not changed.

SUBSTRUCTURE:
The left retaining wall in Span 1 has fallen or been knocked down. There is small rock slope protection (RSP) that has been placed just below were the wall was standing, adjacent to the upstream side of Pier 2. This condition and corrective action provided was first documented in the 2007 report. No further action is required at this time.

There is 58" of vertical exposure of the pile cap at Bent 3. During this inspection no undermining was observed. No corrective action is required at this time.

The BIR dated 05/09/2008 determined this structure is Scour Critical (NBI Item 113 code of 3). A Scour Plan of Action dated 11/21/2008 has been completed. On this date the critical elevations outlined in the scour plan of action were checked and no significant (+/- 6") differences were noted.

SAFE LOAD CAPACITY
A Load Rating Summary Sheet dated 2/11/2010 is on file for this structure and is based on

INSPECTION COMMENTARY

hand calculations. While this inspection does not include a check of that analysis, it does verify that the structural conditions observed during this inspection are consistent with those assumed in that analysis.

OPERATIONAL SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

Elem No.	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
					Qty in each Condition State				
12	Concrete Deck - Bare	2	670	sq.m.	0	670	0	0	0
110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
220	Reinforced Conc Submerged Pile Cap/Footing	2	1	ea.	1	0	0	0	0
228	Timber Submerged Pile	2	1	ea.	1	0	0	0	0
339	Concrete Railing (aesthetic/masonry)	2	183	m.	0	100	83	0	0
358	Deck Cracking	2	1	ea.	0	0	0	1	0
359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
360	Settlement	2	1	ea.	0	1	0	0	0
361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

RecDate: 10/23/2009
Action : Super-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:

Clean and paint the exposed rebar to prevent further deterioration.

RecDate: 10/23/2009
Action : Deck-Methacrylate
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:

Apply methacrylate, or another suitable substance, to seal the deck to prevent water from accelerating the corrosion of the deck reinforcement.

A call was made and an email was sent on July 2, 2013 to the Engineering Manager at Solano County Public works, Matt Tuggle (707-784-6072), in an attempt to determine if any work is scheduled for this structure. As of July 17, 2013, when this report was finalized, the county has not responded to this request for information.

Team Leader : W. John Baker

Report Author : W. John Baker

Inspected By : W.Baker/D.Ambriz

7/17/13

W. John Baker (Registered Civil Engineer) (Date)

REGISTERED PROFESSIONAL ENGINEER
W. John Baker
No. 60307
06/30/2014
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **23C0092**
(5) INVENTORY ROUTE(ON/UNDER)- ON 140000000
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED- PUTAH CREEK
(7) FACILITY CARRIED- STEVENSON BR RD
(9) LOCATION- SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK- NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 13 SEC
(17) LONGITUDE 121 DEG 51 MIN 03 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- ARCH - THRU CODE 212
(44) STRUCTURE TYPE APPR:MATERIAL- CONCRETE CONT
TYPE- TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- NONE CODE 0
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 789
(30) YEAR OF ADT 2008 (109) TRUCK ADT 5 %
(19) BYPASS, DETOUR LENGTH 19 KM

*************** GEOMETRIC DATA ****************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0.2 M RIGHT 0.2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 46.0
STATUS STRUCTURALLY DEFICIENT
HEALTH INDEX 86.4
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- NOT ON NHS 0
(26) FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY- NOT STRAHNET 0
(101) PARALLEL STRUCTURE- NONE EXISTS N
(102) DIRECTION OF TRAFFIC- 2 WAY 2
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- COUNTY HIGHWAY AGENCY 02
(22) OWNER- COUNTY HIGHWAY AGENCY 02
(37) HISTORICAL SIGNIFICANCE- ELIGIBLE 2

**************** CONDITION **************** CODE

(58) DECK 3
(59) SUPERSTRUCTURE 6
(60) SUBSTRUCTURE 5
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- UNKNOWN 0
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 40.8
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 24.3
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 5
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0 0N0
(113) SCOUR CRITICAL BRIDGES 3

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $1,541,000
(95) ROADWAY IMPROVEMENT COST $308,200
(96) TOTAL PROJECT COST $2,588,880
(97) YEAR OF IMPROVEMENT COST ESTIMATE 2010
(114) FUTURE ADT 1518
(115) YEAR OF FUTURE ADT 2035

*************** INSPECTIONS ***************

(90) INSPECTION DATE 03/13 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

Photo No. 1
Damage to the left rail in Span 1

Photo No. 2
Crack on the left rail at the connection with the northern arch over Bent 3

Photo No. 3
Spall with exposed rebar on the southern arch in Span 2 at the 5th column on the left

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 06/30/2011

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers which are founded RC piles. RC, seat type abutments which are founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: UNKNOWN
Inventory Rating: 24.3 metric tonnes Calculation Method: LOAD FACTOR
Operating Rating: 40.8 metric tonnes Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3: Legal Type 3S2: Legal Type 3-3: Legal

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m Net Width: 6.1 m No. of Lanes: 2
Rail Description: Concrete. Rail Code : 0000
Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

INSPECTION ACCESS

Water was flowing under Span 2 during this inspection. All elements were accessible and inspected.

REVISIONS

The routine roadway and elevation photos were updated (photos 1 & 2).

CONDITION OF STRUCTURE
DECK AND RAIL:
There are numerous random spalls and incipient spall in both bridge rails. Many of the spalls have been patched; however, the patches are beginning to fail (break up and spall). These spalls are not structurally significant and require no corrective action.

There is a full width transverse deck crack in Span 1 near Bent 2 which was first documented in the 2007 report. This cracks measures 0.4" wide at the top of the right exterior girder and 0.6" wide at the top of the left exterior girder. There is also a similar transverse deck crack in Span 4 near Bent 4 which is not as sever as the Span 1 crack. This cracking appears to have been caused by settlement of the abutments which are founded on spread footings.

CONDITION TEXT

The deck has transverse cracks spaced between 4'-8' OC with edge spalling measuring up to 1/4" wide. These cracks appear to correspond with the locations of the floor beams. This condition was first documented during the 1990 inspection.

Based on the photos in the last (October 2009) report, the condition of the deck and rail have remained the same.

SUPERSTRUCTURE:
There are minor cracks and spalls in the arch members which have been documented as far back as the 1993 report. These conditions were caused by vehicle impacts as well as a lack of cover over the rebar. At this time the current severity of this condition does not require analysis or corrective action.

There are 2 transverse soffit cracks in Span 1 near Bent 2. There are also 2 transverse soffit cracks in Span 4 near Bent 4. These cracks are reflective of the cracking noted in the deck at these locations. In the areas surrounding these cracks there is water staining as wells as locations of efflorescence and brown staining. This provides visual evidence that water is seeping through the slab.

There are spalls in the girders with exposed corroding rebar.

Numerous soffit spalls are on either side of the structure of all spans. The spalls are typically between 1'-2' in length and are 6" wide with exposed corroding rebar.

The spalling conditions noted above appear to have been caused by a lack of cover over the steel during construction. It is recommended that the exposed rebar be cleaned and painted to prevent further deterioration of the steel. Based on the photos in the last (October 2009) report, the condition of the superstructure has not changed.

SUBSTRUCTURE:
The left retaining wall in Span 1 has fallen or been knocked down. There is small rock slope protection (RSP) that has been placed just below were the wall was standing, adjacent to the upstream side of Pier 2. This condition and corrective action provided was first documented in the 2007 report. No further action is required at this time.

SCOUR
There is 58" of vertical exposure of the pile cap at Bent 3. During this inspection no undermining was observed. No corrective action is required at this time.

The BIR dated 05/09/2008 determined this structure is Scour Critical (NBI Item 113 code of 3). A Scour Plan of Action dated 11/21/2008 has been completed. On this date the critical elevations outlined in the scour plan of action were checked and no significant (+/- 6") differences were noted.

SAFE LOAD CAPACITY

Load ratings were calculated for both the main unit (arch, floor beam, and exterior girder) and the approach span (T-beam) in July 1978. Based on these calculations it was determined the main unit floor beam is the controlling element. The Load Factor floor beam calculations reflect 0" of AC on the deck. These calculations yielded the structure capable of sustaining all State legal and permit truck loads.

Inventory Rating = 24.3 metric tons, Rating Factor: 0.75
Operating Rating = 40.8 metric tons, Rating Factor: 1.26
Permit ratings: PPPPP, Rating Factor = 1.01

SIGNS

CONDITION TEXT

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

Elem No.	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4	St. 5
12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	0
358	Deck Cracking	2	1	ea.	0	0	0	1	0
359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
360	Settlement	2	1	ea.	0	1	0	0	0
361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

RecDate: 10/23/2009
Action : Super-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:

Clean and paint the exposed rebar to prevent further deterioration.

RecDate: 10/23/2009
Action : Deck-Methacrylate
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget: 2 YEARS
DistTarget:
EA:

Apply methacrylate, or another suitable substance, to seal the deck to prevent water from accelerating the corrosion of the deck reinforcement.

Inspected By : W. John Baker

W. John Baker (Registered Civil Engineer)

REGISTERED PROFESSIONAL ENGINEER
W. John Baker
No. 60307
06/30/2012
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1)	STATE NAME- CALIFORNIA	069
(8)	STRUCTURE NUMBER	23C0092
(5)	INVENTORY ROUTE(ON/UNDER)- ON	1400W8510
(2)	HIGHWAY AGENCY DISTRICT	04
(3)	COUNTY CODE 095 (4) PLACE CODE	00000
(6)	FEATURE INTERSECTED-	PUTAH CREEK
(7)	FACILITY CARRIED-	STEVENSON BR RD
(9)	LOCATION-	SOL/YOL CO LINE
(11)	MILEPOINT/KILOMETERPOINT	0
(12)	BASE HIGHWAY NETWORK- NOT ON NET	0
(13)	LRS INVENTORY ROUTE & SUBROUTE	
(16)	LATITUDE	38 DEG 32 MIN 13 SEC
(17)	LONGITUDE	121 DEG 51 MIN 03 SEC
(98)	BORDER BRIDGE STATE CODE % SHARE	%
(99)	BORDER BRIDGE STRUCTURE NUMBER	

******** STRUCTURE TYPE AND MATERIAL *********

(43)	STRUCTURE TYPE MAIN:MATERIAL-	CONCRETE CONT
	TYPE- ARCH - THRU	CODE 212
(44)	STRUCTURE TYPE APPR:MATERIAL-	CONCRETE CONT
	TYPE- TEE BEAM	CODE 204
(45)	NUMBER OF SPANS IN MAIN UNIT	2
(46)	NUMBER OF APPROACH SPANS	2
(107)	DECK STRUCTURE TYPE- CIP CONCRETE	CODE 1
(108)	WEARING SURFACE / PROTECTIVE SYSTEM:	
A)	TYPE OF WEARING SURFACE- NONE	CODE 0
B)	TYPE OF MEMBRANE- NONE	CODE 0
C)	TYPE OF DECK PROTECTION- NONE	CODE 0

*************** AGE AND SERVICE ***************

(27)	YEAR BUILT	1923
(106)	YEAR RECONSTRUCTED	0000
(42)	TYPE OF SERVICE: ON- HIGHWAY	1
	UNDER- WATERWAY	5
(28)	LANES:ON STRUCTURE 02 UNDER STRUCTURE	00
(29)	AVERAGE DAILY TRAFFIC	789
(30)	YEAR OF ADT 2008 (109) TRUCK ADT	5 %
(19)	BYPASS, DETOUR LENGTH	19 KM

*************** GEOMETRIC DATA ***************

(48)	LENGTH OF MAXIMUM SPAN	32.9 M
(49)	STRUCTURE LENGTH	90.8 M
(50)	CURB OR SIDEWALK: LEFT 0.2 M	RIGHT 0.2 M
(51)	BRIDGE ROADWAY WIDTH CURB TO CURB	6.1 M
(52)	DECK WIDTH OUT TO OUT	7.1 M
(32)	APPROACH ROADWAY WIDTH (W/SHOULDERS)	5.8 M
(33)	BRIDGE MEDIAN- NO MEDIAN	0
(34)	SKEW 0 DEG (35) STRUCTURE FLARED	NO
(10)	INVENTORY ROUTE MIN VERT CLEAR	4.31 M
(47)	INVENTORY ROUTE TOTAL HORIZ CLEAR	6.1 M
(53)	MIN VERT CLEAR OVER BRIDGE RDWY	4.31 M
(54)	MIN VERT UNDERCLEAR REF- NOT H/RR	0.00 M
(55)	MIN LAT UNDERCLEAR RT REF- NOT H/RR	0.0 M
(56)	MIN LAT UNDERCLEAR LT	0.0 M

*************** NAVIGATION DATA ***************

(38)	NAVIGATION CONTROL- NOT APPLICABLE	CODE N
(111)	PIER PROTECTION-	CODE
(39)	NAVIGATION VERTICAL CLEARANCE	0.0 M
(116)	VERT-LIFT BRIDGE NAV MIN VERT CLEAR	M
(40)	NAVIGATION HORIZONTAL CLEARANCE	0.0 M

****

SUFFICIENCY RATING =	45.0
STATUS	STRUCTURALLY DEFICIENT
HEALTH INDEX	85.1
PAINT CONDITION INDEX =	N/A

************* CLASSIFICATION ************* CODE

(112)	NBIS BRIDGE LENGTH- YES	Y
(104)	HIGHWAY SYSTEM- NOT ON NHS	0
(26)	FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL	07
(100)	DEFENSE HIGHWAY- NOT STRAHNET	0
(101)	PARALLEL STRUCTURE- NONE EXISTS	N
(102)	DIRECTION OF TRAFFIC- 2 WAY	2
(103)	TEMPORARY STRUCTURE-	
(105)	FED.LANDS HWY- NOT APPLICABLE	0
(110)	DESIGNATED NATIONAL NETWORK - NOT ON NET	0
(20)	TOLL- ON FREE ROAD	3
(21)	MAINTAIN- COUNTY HIGHWAY AGENCY	02
(22)	OWNER- COUNTY HIGHWAY AGENCY	02
(37)	HISTORICAL SIGNIFICANCE- ELIGIBLE	2

**************** CONDITION **************** CODE

(58)	DECK	3
(59)	SUPERSTRUCTURE	6
(60)	SUBSTRUCTURE	5
(61)	CHANNEL & CHANNEL PROTECTION	6
(62)	CULVERTS	N

********* LOAD RATING AND POSTING ********* CODE

(31)	DESIGN LOAD- UNKNOWN	0
(63)	OPERATING RATING METHOD- LOAD FACTOR	1
(64)	OPERATING RATING-	40.8
(65)	INVENTORY RATING METHOD- LOAD FACTOR	1
(66)	INVENTORY RATING-	24.3
(70)	BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS	5
(41)	STRUCTURE OPEN, POSTED OR CLOSED-	A
	DESCRIPTION- OPEN, NO RESTRICTION	

**************** APPRAISAL **************** CODE

(67)	STRUCTURAL EVALUATION	5
(68)	DECK GEOMETRY	3
(69)	UNDERCLEARANCES, VERTICAL & HORIZONTAL	N
(71)	WATER ADEQUACY	7
(72)	APPROACH ROADWAY ALIGNMENT	3
(36)	TRAFFIC SAFETY FEATURES	0000
(113)	SCOUR CRITICAL BRIDGES	3

********** PROPOSED IMPROVEMENTS **********

(75)	TYPE OF WORK- DECK REHABILITATION CODE	36
(76)	LENGTH OF STRUCTURE IMPROVEMENT	90.8 M
(94)	BRIDGE IMPROVEMENT COST	$1,541,000
(95)	ROADWAY IMPROVEMENT COST	$308,200
(96)	TOTAL PROJECT COST	$2,588,880
(97)	YEAR OF IMPROVEMENT COST ESTIMATE	2010
(114)	FUTURE ADT	1488
(115)	YEAR OF FUTURE ADT	2029

*************** INSPECTIONS ***************

(90)	INSPECTION DATE 06/11 (91) FREQUENCY		24 MO	
(92)	CRITICAL FEATURE INSPECTION:			(93) CFI DATE
A)	FRACTURE CRIT DETAIL-	NO	MO	A)
B)	UNDERWATER INSP-	NO	MO	B)
C)	OTHER SPECIAL INSP-	NO	MO	C)

Photo No. 1
Roadway view looking North

Photo No. 2
Elevation view looking Northeast

Structure Maintenance & Investigations

Structure Rating Summary Sheet

Bridge Number: 23C0092
Facility Carried: STEVENSON BR RD
Location: SOL/YOL CO LINE
City:

Bridge Name: PUTAH CREEK

Structural Element Rated: 1923 - 4 Span structure
Spans 1 and 4 - T-beam
Spans 2 and 4 - Tied Arch

Rating Summary

DESIGN LOADING

	Rating Factor	Metric Tonnes	Structure	Control Element	Load Action	Location
Inventory:	0.75	24.3	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway
Operating:	1.26	40.8	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway

LEGAL RATING

		Posting U.S. Tons				
Type 3 (25T):						
Type 3S2 (36T):						
Type 3-3 (40T):						

PERMIT RATING

		Permit Rating				
5 Axle Truck :	1.01	P	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway
7 Axle Truck :	1.01	P	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway
9 Axle Truck :	1.01	P	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway
11 Axle Truck:	1.01	P	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway
13 Axle Truck:	1.01	P	Main Unit (Span 2 and 3)	Floorbeam	Ultimate Moment	Centerline of roadway

RELEVANT LOAD RATING INFORMATION

NOTES:

Load ratings were calculated for both the main unit (arch, floor beam, and exterior girder) and the approach span (T-beam) in July 1978. Based on these calculations it was determined the main unit floor beam is the controlling element. The Load Factor floor beam calculations reflect 0" of AC on the deck.

Overlay Used In Rating: 0.0"

Rating Method: Load Factor (LF) Inventory (65) — Load Factor (LF) Operating (63)

Analysis Tool Used: Hand Calculations

Rating/File Location: Bridge Book

Control Rating By: T. Tsukiji — Rating Date: 07/21/1978

Rating Checked By: SM&I

Rating Type: Calculated

Summary Prepared By: Nick Semander — Summary Date: 02/11/2010

REGISTERED PROFESSIONAL ENGINEER
W. JOHN BAKER
No. C 60307
Exp. 06/30/10
CIVIL
STATE OF CALIFORNIA

W. John Baker - Registered Engineer

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number	: 23C0092
Facility Carried	: STEVENSON BR RD
Location	: SOL/YOL CO LINE
City	:
Inspection Date	: 10/23/2009

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built :	1923	Skew (degrees):	0
Year Widened:	N/A	No. of Joints :	0
Length (m) :	90.8	No. of Hinges :	0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers which are founded RC piles. RC, seat type abutments which are founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: OTHER OR UNKNOWN
Inventory Rating: 24.3 metric tonnes — Calculation Method: LOAD FACTOR
Operating Rating: 40.8 metric tonnes — Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3: Legal — Type 3S2: Legal — Type 3-3: Legal

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m — Net Width: 6.1 m — No. of Lanes: 2
Rail Description: Concrete. — Rail Code : 0000
Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

ACCESS

Water was flowing under Span 2 during this inspection. All elements were accessible and inspected.

REVISIONS

NBI #108A (Type of Wearing Surface) is changed from Concrete to None because no additional concrete was added for a wearing surface.

CONDITION OF STRUCTURE

DECK AND RAIL:
There are numerous random spalls and incipient spall in both bridge rails. Many of the spalls have been patched; however, the patches are beginning to fail (break up and spall). These spalls are not structurally significant and require no corrective action. See attached photos.

There is a full width transverse deck crack in Span 1 near Bent 2 which was first documented in the 2007 report. This cracks measures 0.4" wide at the top of the right exterior girder and 0.6" wide at the top of the left exterior girder. There is also a similar transverse deck crack in Span 4 near Bent 4 which is not as sever as the Span 1

CONDITION TEXT

crack. This cracking appears to have been caused by settlement of the abutments which are founded on spread footings. See attached photos.

The deck has transverse cracks spaced between 4'-8' OC with edge spalling measuring up to 1/4" wide. These cracks appear to correspond with the locations of the floor beams. This condition was first documented 1990 and since that time the spacing has decreased from 8'-12' (as noted in the 2000 repot). See attached photos.

It is recommended that the deck be sealed with methacrylate to prevent water from accelerating the corrosion of the deck reinforcement.

SUPERSTRUCTURE:

There are minor cracks and spalls in the arch members which have been documented as far back as the 1993 report. These conditions were caused by vehicle impacts as well as a lack of cover over the rebar. At this time the current severity of this condition does not require analysis or corrective action. See attached photos.

There are 2 transverse soffit cracks in Span 1 near Bent 2. There are also 2 transverse soffit cracks in Span 4 near Bent 4. These cracks are reflective of the cracking noted in the deck at these locations. In the areas surrounding these cracks there is water staining as wells as locations of efflorescence and brown staining. This provides visual evidence that water is seeping through the slab. See attached photos.

There are spalls in the girders with exposed corroding rebar. See attached photo.

Numerous soffit spalls are on either side of the structure of all spans. The spalls are typically between 1'-2' in length and are 6" wide with exposed corroding rebar. See attached photos.

The spalling conditions noted above appear to have been caused by a lack of cover over the steel during construction. It is recommended that the exposed rebar be cleaned and painted to prevent further deterioration of the steel.

SUBSTRUCTURE:

The left retaining wall in Span 1 has fallen or been knocked down. There is small rock slope protection (RSP) that has been placed just below were the wall was standing, adjacent to the upstream side of Pier 2. This condition and corrective action provided was first documented in the 2007 report. No further action is required at this time. See attached photo.

SCOUR

There is 58" of vertical exposure of the pile cap at Bent 3. During this inspection no undermining was observed. No corrective action is required at this time. See attached photo.

The BIR dated 05/09/2008 determined this structure is Scour Critical (NBI Item 113 code of 3). A Scour Plan of Action dated 11/21/2008 has been completed. On this date the critical elevations outlined in the scour plan of action were checked and no significant differences were noted.

SAFE LOAD CAPACITY

Load ratings were calculated for both the main unit (arch, floor beam, and exterior girder) and the approach span (T-beam) in July 1978. Based on these calculations it was determined the main unit floor beam is the controlling element. The Load Factor floor beam calculations reflect 0" of AC on the deck. These calculations yielded the structure capable of sustaining all State legal and permit truck loads.

CONDITION TEXT

Inventory Rating = 24.3 metric tons, Rating Factor: 0.75
Operating Rating = 40.8 metric tons, Rating Factor: 1.26
Permit ratings: PPPPP, Rating Factor = 1.01

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

F#	Elem	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
101	12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
101	110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
101	144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
101	155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
101	205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
101	215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
101	331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	0
101	358	Deck Cracking	2	1	ea.	0	0	0	1	0
101	359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
101	360	Settlement	2	1	ea.	0	1	0	0	0
101	361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

RecDate: 10/23/2009
Action : Super-Patch spalls
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost: $
StrTarget: 2 YEARS
DistTarget:
EA:

Clean and paint the exposed rebar to prevent further deterioration.

RecDate: 10/23/2009
Action : Deck-Methacrylate
Work By: LOCAL AGENCY
Status : PROPOSED
EstCost: $
StrTarget: 2 YEARS
DistTarget:
EA:

Apply methacrylate, or another suitable substance, to seal the deck to prevent water from accelerating the corrosion of the deck reinforcement.

Inspected By : W.Baker/N.Semander

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
W. JOHN BAKER
No. C 60307
Exp. 06/30/10
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

*************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **23C0092**
(5) INVENTORY ROUTE(ON/UNDER)- ON 1400W8510
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED- PUTAH CREEK
(7) FACILITY CARRIED- STEVENSON BR RD
(9) LOCATION- SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK- NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 13 SEC
(17) LONGITUDE 121 DEG 51 MIN 03 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- ARCH - THRU CODE 212
(44) STRUCTURE TYPE APPR:MATERIAL- CONCRETE CONT
TYPE- TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- NONE CODE 0
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 900
(30) YEAR OF ADT 1993 (109) TRUCK ADT 5 %
(19) BYPASS, DETOUR LENGTH 19 KM

*************** GEOMETRIC DATA ****************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0.2 M RIGHT 0.2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 44.8
STATUS STRUCTURALLY DEFICIENT
HEALTH INDEX 85.1
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- NOT ON NHS 0
(26) FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY- NOT STRAHNET 0
(101) PARALLEL STRUCTURE- NONE EXISTS N
(102) DIRECTION OF TRAFFIC- 2 WAY 2
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- COUNTY HIGHWAY AGENCY 02
(22) OWNER- COUNTY HIGHWAY AGENCY 02
(37) HISTORICAL SIGNIFICANCE- ELIGIBLE 2

**************** CONDITION **************** CODE

(58) DECK 3
(59) SUPERSTRUCTURE 6
(60) SUBSTRUCTURE 5
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- OTHER OR UNKNOWN 0
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 40.8
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 24.3
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 5
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0000
(113) SCOUR CRITICAL BRIDGES 3

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $1,541,000
(95) ROADWAY IMPROVEMENT COST $308,200
(96) TOTAL PROJECT COST $2,588,880
(97) YEAR OF IMPROVEMENT COST ESTIMATE 2010
(114) FUTURE ADT 1488
(115) YEAR OF FUTURE ADT 2029

*************** INSPECTIONS ***************

(90) INSPECTION DATE 10/09 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

BRIDGE SCOUR EVALUATION - PLAN OF ACTION				
Br. No. 23C0092	**Owner** Solano County	**Location** SOL/YOL CO LINE	**Facility Carried** STEVENSON BR ROAD	**Name** PUTAH CREEK

Plan of Action Completed By: Robert Liu, PE	**Date of Completion: 11-21-08**

1. SCOUR VULNERABILITY RATING

Scour Evaluation Summary:
This bridge was classified as NBIS Item 113 Code 3 – from a numerical analysis that was performed by Caltrans stating that the bridge foundations are determined to be unstable for assessed or calculated scour conditions: - scour within the limits of footing/piling, scour below the spread-footing base.

Scour History:
Bridge reports performed between 7/9/71 and 4/6/82 indicate exposure of approximately 3 feet of timber piles below the pile cap at pier 3. A report from 4/26/88 showed the footing was protected with a blanket of riprap. The latest report on 5/2/08 verified the riprap and upstream channel cross-section. The cross-section of the channel has remained stable since 1971.

a. **Foundation Type** ☐ Spread footing ☐ Pile Extension ☒ Footing on Piles ☐ Unknown

b. **Foundation Material** ☒ Known ___sand/gravel sand_____ ☐ Unknown

Scour Review: Done By: Caltrans, Charles J. Ineichen Date: 5-9-08

Structural Assessment: Done By: Caltrans, Charles J. Ineichen Date: 5-9-08
Critical Elevation: ________________

Geotechnical Assessment: Done By: Date:
Critical Elevation: ________________

2. NBIS CODING INFORMATION

	Most Recent
Inspection date	5/8/08
Item 113 Scour	3

3. COUNTERMEASURE RECOMMENDATION

A. Completed Countermeasures:
County inspection was performed on September 22, 2008 to verify May 9th 2008 report.

B. Proposed Countermeasures:

The channel has remained relatively stable since the 1971 bridge report and further degradation of the channel is not anticipated, however, County forces will monitor the bridge when the flow rate in the creek exceeds 4,500 cfs or about 10 feet above the pile. This flow is sufficient to cause potential scour at the pile cap. Additionally, an annual inspection will be performed to check for signs of degradation, undermining of main channel and footing riprap.

Solano County Water Agency flow records at Stevenson Bridge show that at an approximate 14-foot water depth the measured water flow is 5400 cfs.

The maximum water flow since construction of the Monticello Dam upstream is approximately 24 feet deep with an approximate flow rate of 17,000 cfs.

☒ **Countermeasures Not Required. (Please explain)**
Riprap installed in the 1980's has remained stable and the channel cross-section has remained stable. The bridge is scheduled for rehab in about 5 years.

☐ **Install Scour Countermeasures** (See 4 and 5)	Estimated Cost
___Riprap with monitoring program	$
___Guide bank	$
___Spurs / Bendway weirs / Barbs	$
___Relief bridge / Culvert	$
___Channel improvements	$
___Monitoring	$
___Monitoring device	$
___Check Dam	$
___Substructure Modification	$
___Bridge replacement	$
___Other ______________________________	$

☐ **Close Bridge** (See 6)

C.

4. COUNTERMEASURE IMPLEMENTATION SCHEDULE

Countermeasure Implementation Project Type:

☐ Proposed Construction Project

Lead Agency

☐ Maintenance Project

Advertised Date:

Other scheduling information:

5. MONITORING PLAN

Monitoring Plan Summary:

County forces will continue monitoring for potential scour after each event that exceeds a flow rate of 4,500 cfs or approximately 10 feet above the pile cap by measuring for any settlement of the pile cap and by visual inspection for scour as soon as practical after each event.

Flow rates can be verified at the Solano County Water Agency web site at:

https://www.grabdata.com/assets/SCWA/pcsteve_lvl.htm

Monitoring Authority:

☒ **Regular Inspection Program of __12_ mo.** ☐ w/surveyed cross sections
Items to Watch: riprap around the pier 3 footing

☐ **Increased Inspection Interval of ______ mo.** ☐ w/surveyed cross sections
Items to Watch:

☐ **Underwater Inspection Program** **Frequency ______ mo.**
Items to Watch:

☐ **Fixed Monitoring Device**
Type of Instrument:
Installation location(s):
Sample Interval: ☐ 30 min. ☐ 1 hr. ☐ 6 hrs. ☐ 12 hrs. ☐ Other ________
Frequency of data logger downloading: ☐ Weekly ☐ Bi-weekly ☐ Monthly ☐ Other ________
Scour-critical discharge:________
Action required if scour-critical elevation detected:

☒ **Other Monitoring Program**
Type: ☒ Visual
☐ Instrument
☐ Portable ☐ Geophysical ☐ Sonar
☐ Other gages

Flood monitoring required: ☒ Yes ☐ No
Flood monitoring event defined by:
☒ Discharge over _4500 cubic feet / sec from SCWA website
☐ Stage ______
☒ Elev. measured from 10' above pile cap (correlates with 4500cfs)

Frequency of flood monitoring: ☐ 1 hr. ☐ 3 hr. ☐ 6 hrs. ☐ Other ______
Scour critical elevation: n/a
Action required if scour-critical elevation detected: Monitor bridge for signs of settlement from established bench marks and close the bridge if settlement is greater than 1/2".

6. BRIDGE CLOSURE PLAN

Bridge ADT: 535	Built: 1923	5 % Trucks:	Bridge Length (ft): 90.8

Once a flow rate has reached 4,500 cfs, a daily elevation survey of the structure will be conducted. Bench marks have been placed, see attached drawing, and if a settlement of 1/2" or greater is detected the bridge will be closed. After the water recedes, we will inspect the bridge and determine if additional mitigation is required before permitting traffic on the structure.

Scour Monitoring Criteria for Consideration of Bridge Closure:

- ☐ Water surface elevation reaches ______________________
- ☒ Overtopping road or structure
- ☐ Scour Measurement Results / Monitoring Device ☒ Loss of Riprap
- ☒ Observed amount of 1/2" settlement of the benchmark located at the bridge midpoint
- ☐ Loss of Road Embankment
- ☐ Debris Accumulation
- ☐ Other ____________

Person / Area Responsible for Closure: Paul Wiese

Contact People (Name & Phone No.): Paul Wiese (707) 784-6072

Responsible for re-opening after inspection: Paul Wiese

7. DETOUR ROUTE

Detour route description (route number, from - to, etc.) – attach map.
If on north side of the bridge – go east on Russell Blvd and south on Pedrick Road. If on south side of the bridge - go east on Sievers to north on Pedrick Road.

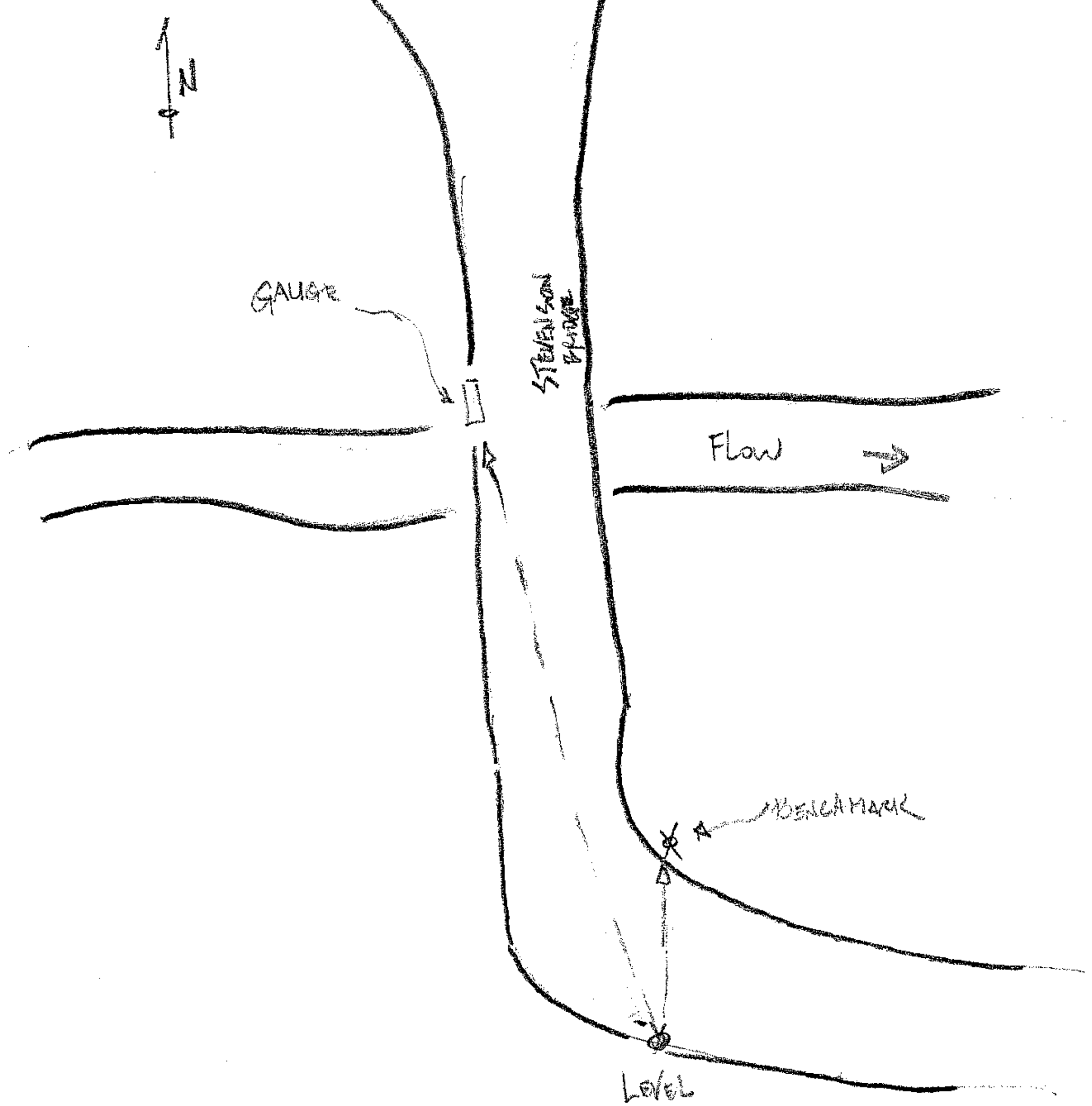
Average ADT: 3567	Year: 1995	5% Trucks:	Length: 98.8

Bridges on Detour Route:

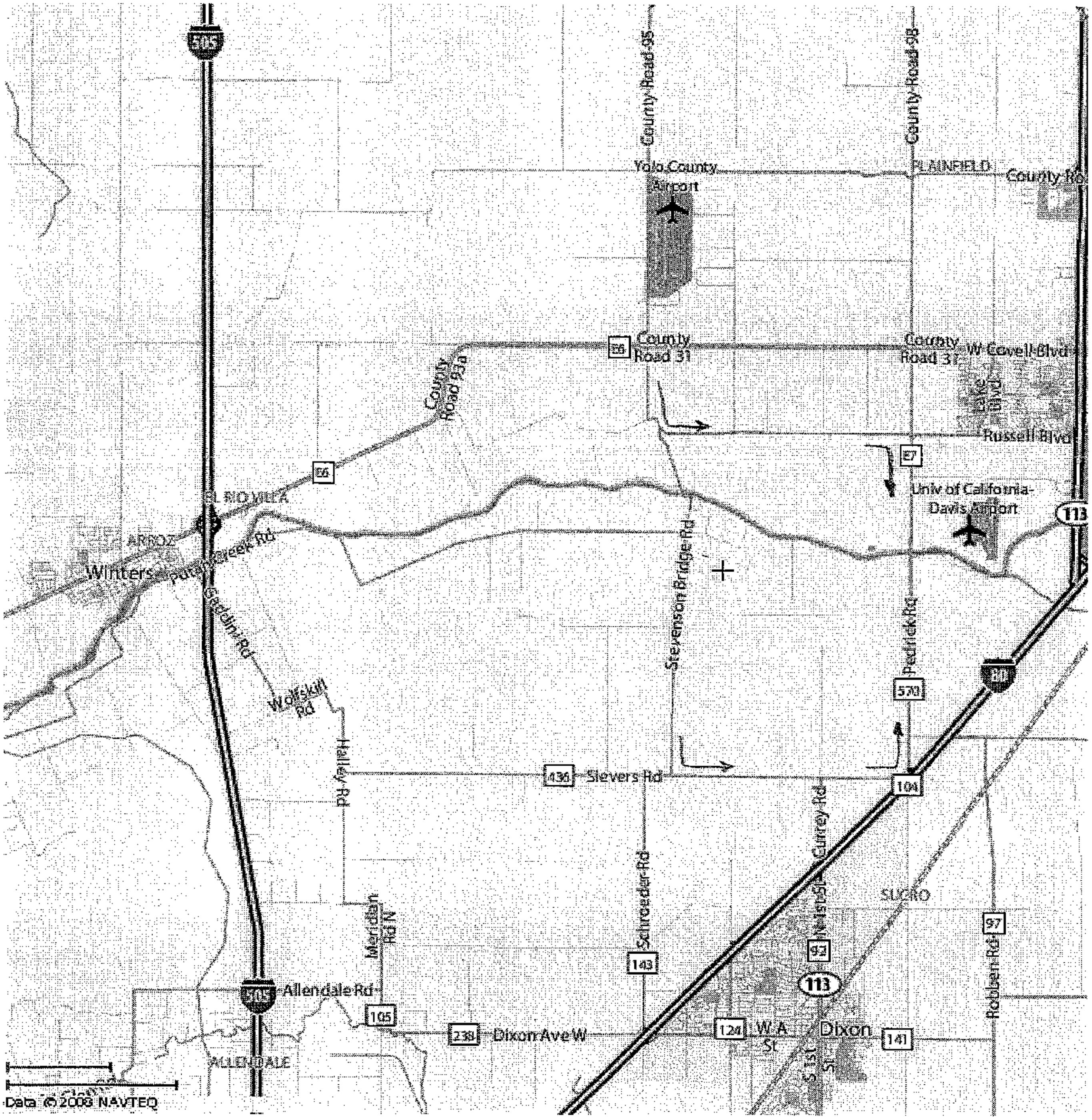
Bridge Number	Waterway	Sufficiency Rating/ Load limitations	Scour 113 code
23C0033	Putah Creek	HS-20	3

STEVENSON ROAD BRIDGE

MONITORING PLAN



YAHOO! LOCAL Maps



DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 05/09/2008

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
☐	☐	☐	☐	X

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers which are founded RC piles. RC, seat type abutments which are founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: OTHER OR UNKNOWN
Inventory Rating: 24.5 metric tons — Calculation Method: LOAD FACTOR
Operating Rating: 40.8 metric tons — Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A — Type 3S2 N/A — Type 3-3 N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m — Net Width: 6.1 m — No. of Lanes: 2
Rail Description: Concrete. — Rail Code : 0000
Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

HISTORY
The 7/9/71 through 4/6/82 bridge reports noted a separation of the retaining wall from the Pier 2 column and indicated that scour is occurring around the piles, exposing approximately 3 feet of timber piles below the footing block at Pier 3. The 4/26/88 report indicated the Pier 3 footing had been protected with a heavy blanket of rocks. The 9/30/97 through 4/27/05 bridge reports referred to the eminent separation case of the retaining wall from the Pier 2 column and exposure of the Pier 3 footing without scouring. The 3/29/07 report indicated the retaining wall in span 1 had fallen or been knocked down and new small rock slope protection (RSP) has been placed just below where the wall was standing.

The structure has been given an Element Level Inspection 361 Code, Scour Smart flag with Condition State of 2: "Scour exists at the bridge site and if left unchecked could adversely impact the structural integrity of the bridge".

REVISION
The National Bridge Inspection Item 113 Code has been revised from U to 3.

SCOUR
This report addresses hydraulic issues only. The structure's scour potential has been assessed in accordance with the FHWA Technical Advisory T5140.23, "Evaluating Scour at Bridges". The NBI Item 113 Code, "Vulnerability to Scour", is changed to 3: "Bridge is scour critical: bridge foundations determined to be unstable for assessed or calculated

CONDITION TEXT

scour conditions; scour within limits of footing or piles ".

Structure Hydraulics conducted a field investigation on 5/2/08 and verified an upstream channel cross-section measurement at the bridge that is attached to the 3/29/07 Bridge Inspection Report (BIR). Comparisons of this latest channel cross-section with two available previous cross-sections taken by the Area Bridge Maintenance Engineer (ABME) in 1971 and 1993 show that the channel bottom has not significantly changed since 1971 and the channel has remained relatively stable.

The channel bed materials looked to be composed of sand and gravel with light growth of grass and some trees. The channel banks were covered with a heavy growth of shrubs and trees along both embankments. The maximum water depth on the day of the investigation was approximately 5 feet in the main waterway. The main waterway was located in Span 2 and the thalweg was located close to Pier 3. The channel appeared well aligned with the bridge opening.

Roadway runoff gullies run down both sides of the Abutment 1 slopes and need proper protection. Minor pile cap exposure was noted on the Span 2 side of Pier 2, but no undermining was observed. No scour or scour potential was noted at either Pier 4 or Abutment 5.

Pier 3 pile cap was exposed up to 4 feet but no undermining was noted. Rock protection placed around the Pier 3 pile cap was noted and appeared to be in adequate condition. However, if the rock protection were to fail, theoretical scour shows a potential for a large scour hole possibly developing at Pier 3 during a significant hydraulic event. Records show that Pier 3 is founded on timber piles. Given the amount of scour potential, Pier 3 could become unstable. Therefore, the structure is considered scour critical at this time.

RECOMMENDATION

We recommend that the local agency investigate the integrity of the pile foundation at Pier 3. In the past, this timber pile group has been exposed to dry wet conditions. The local agency should then provide scour mitigation at the site. Furthermore, a Federal Regulations, 23 Code of Federal Regulation CFR 650 subpart C, requires a Plan of Action (POA) for each scour critical bridge within your jurisdiction. In order to meet the Federal Highway Administration compliance, we recommend that the local agency develop and implement a POA for the subject bridge.

Inspected By : H. Azizi

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
CHARLES J. INEICHEN
No. 054230
Exp. 12-31-09
CIVIL
STATE OF CALIFORNIA

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number	: **23C0092**
Facility Carried:	STEVENSON BR RD
Location	: SOL/YOL CO LINE
City	:
Inspection Date	: 03/29/2007

Inspection Type

Routine	FC	Underwater	Special	Other
X				

Bridge Inspection Report

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built :	1923	Skew (degrees):	0
Year Widened:	N/A	No. of Joints :	0
Length (m) :	90.8	No. of Hinges :	0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers which are founded RC piles. RC, seat type abutments which are founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: OTHER OR UNKNOWN

Inventory Rating:	24.5	metric tons	Calculation Method:	LOAD FACTOR		
Operating Rating:	40.8	metric tons	Calculation Method:	LOAD FACTOR		
Permit Rating :	PPPPP					
Posting Load :	Type 3	N/A	Type 3S2	N/A	Type 3-3	N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r

Total Width: 7.1 m   Net Width: 6.1 m   No. of Lanes: 2

Rail Description: Concrete.   Rail Code : 0000

Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

CONDITION OF STRUCTURE

Crack size and density:
Light: Less than 0.02"
Moderate: 0.02" to 0.08" and spacing of 1 foot of greater
Severe: Greater than 0.08" and spacing of less than 1 foot

The random spalls in the bridge rail have been patched, although it appears the patches are beginning to break up and spall.

The retaining wall in span 1 has fallen or been knocked down. There is new small rock slope protection (RSP) that has been placed just below were the wall was standing, adjacent to the upstream side of pier 2.

In span 1 there is a severe transverse deck crack the reflects through to the soffit and extends out to both exterior girders. The crack measures 0.4 inches wide at the top of the right exterior girder and 0.6 inches wide at the top of the left exterior girder. It appears as though the crack has previously been patched, however it has opened back up again.

In span 2 there are a few spalls on the left and right sides of the soffit with exposed

CONDITION TEXT

longitudinal rebars.

In span 3 there are several shallow spalls exposing longitudinal rebar on the left and right edges of the soffit just inside of the exterior girders. The right exterior girder, just past half span, has a spall with three exposed rebars.

In span 4, bay 1 at abutment 5 there is a spall with one exposed transverse rebar that is corroding. In bay 4 near pier 4 there are several spalls with exposed transverse rebar.

The following conditions have been noted in the previous investigation and were compared to the observations made in the field. Based on the condition state language there appears to be no significant changes to the following conditions:

As noted in previous reports, the deck has large transverse deck cracks approximately 10 to 12 feet on center which appear to correspond with the locations of the floor beams.

The minor cracks and spalling in the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The girders have a few spalls with rebar exposed. Previous patches are starting to fall out.

A Stream cross section was taken at the time of this inspection.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

F#Elem	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
					Qty in each Condition State				
101 12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
101 110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
101 144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
101 155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
101 205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
101 215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
101 331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	0
101 358	Deck Cracking	2	1	ea.	0	0	0	1	0
101 359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
101 360	Settlement	2	1	ea.	0	1	0	0	0
101 361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS - NONE

CHANNEL X-SECTION

Side : Upstream X-Section Date: 03/29/2007

Measured From :Top of Concrete Rail

Location	Horiz(m)	Vert(m)	Comments
Abutment 1	1.00	5.10	
	7.00	8.30	
	7.40	11.00	

CHANNEL X-SECTION

Side : Upstream

X-Section Date: 03/29/2007

Measured From : Top of Concrete Rail

Location	Horiz(m)	Vert(m)	Comments
	8.30	12.00	
	14.00	14.00	
	26.40	15.20	
	30.00	16.80	Thalweg
	40.00	15.30	Span 2 sid e of pier 3 pile cap
	40.30	14.10	Top of pier 3 pile cap
	41.50	15.50	center line of pier 3 pile cap
	58.00	12.80	
	74.90	7.10	center line pier 4
Abutment 5	85.70	5.00	

Inspected By : W. John Baker

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
W. JOHN BAKER
No. C 60307
Exp. 06/30/08
CIVIL
STATE OF CALIFORNIA

8/24/07

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **23C0092**
(5) INVENTORY ROUTE(ON/UNDER)- ON 1400W8510
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED- PUTAH CREEK
(7) FACILITY CARRIED- STEVENSON BR RD
(9) LOCATION- SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK- NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 13 SEC
(17) LONGITUDE 121 DEG 51 MIN 03 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- ARCH - THRU CODE 212
(44) STRUCTURE TYPE APPR:MATERIAL- CONCRETE CONT
TYPE- TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- CONCRETE CODE 1
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 900
(30) YEAR OF ADT 1993 (109) TRUCK ADT 5 %
(19) BYPASS, DETOUR LENGTH 19 KM

*************** GEOMETRIC DATA ****************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0.2 M RIGHT 0.2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 45.1
STATUS STRUCTURALLY DEFICIENT
HEALTH INDEX 85.1
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- NOT ON NHS 0
(26) FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY- NOT STRAHNET 0
(101) PARALLEL STRUCTURE- NONE EXISTS N
(102) DIRECTION OF TRAFFIC- 2 WAY 2
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- COUNTY HIGHWAY AGENCY 02
(22) OWNER- COUNTY HIGHWAY AGENCY 02
(37) HISTORICAL SIGNIFICANCE- ELIGIBLE 2

**************** CONDITION **************** CODE

(58) DECK 3
(59) SUPERSTRUCTURE 6
(60) SUBSTRUCTURE 5
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- OTHER OR UNKNOWN 0
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 40.8
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 24.5
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 5
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0000
(113) SCOUR CRITICAL BRIDGES U

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $336,000
(95) ROADWAY IMPROVEMENT COST $34,000
(96) TOTAL PROJECT COST $504,000
(97) YEAR OF IMPROVEMENT COST ESTIMATE 1999
(114) FUTURE ADT 1420
(115) YEAR OF FUTURE ADT 2015

*************** INSPECTIONS ***************

(90) INSPECTION DATE 03/07 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 04/27/2005

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers and RC piles, seat abutments. All founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: OTHER OR UNKNOWN
Inventory Rating: 24.5 metric tons Calculation Method: LOAD FACTOR
Operating Rating: 40.8 metric tons Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A Type 3S2 N/A Type 3-3 N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m Net Width: 6.1 m No. of Lanes: 2
Rail Description: Concrete. Rail Code : 0000
Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

CONDITION OF STRUCTURE

As noted in previous reports, the deck has large transverse deck cracks approximately 3 to 4 m on center which appear to correspond with the locations of the floor beams.

There are large transverse soffit cracks in Spans 1 and 3, full width. Some have rebar exposed with rust staining.

The minor cracks and spalling in the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The retaining wall, which protects the bank in Span 1, has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of the wall has not changed and remains at 300 mm as previously reported.

The girders have a few spalls with rebar exposed. Previous patches are starting to fall out.

The bridge rail has random spalls and delaminations with rebar exposed throughout.

On Span 1, Girder 1 to 3, approximately 2.5 m from Bent 2, there are 3/4 depth, full

CONDITION TEXT

width, severe size (20 mm wide) girder cracks with full thickness deck cracking with severe rust staining and exposed rebar. The deck cracks are patched with AC.

Span 4 exhibits similar cracking throughout all girders with cracking being slightly more severe on the right hand side of the structure.

Stream cross section was not taken due to swift current in the creek at this time of inspection.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

F#Elem	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4	St. 5
01 12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
01 110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
01 144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
01 155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
01 205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
01 215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
01 331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	0
01 358	Deck Cracking	2	1	ea.	0	0	0	1	0
01 359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
01 360	Settlement	2	1	ea.	0	1	0	0	0
01 361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS - NONE

Inspected By : Michael Nguyen

Registered Civil Engineer

STRUCTURE INVENTORY AND APPRAISAL REPORT

*************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **23C0092**
(5) INVENTORY ROUTE(ON/UNDER)- ON 1400W8510
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED- PUTAH CREEK
(7) FACILITY CARRIED- STEVENSON BR RD
(9) LOCATION- SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK- NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 13 SEC
(17) LONGITUDE 121 DEG 51 MIN 03 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL ********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- ARCH - THRU CODE 212
(44) STRUCTURE TYPE APPR:MATERIAL- CONCRETE CONT
TYPE- TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- CONCRETE CODE 1
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 900
(30) YEAR OF ADT 1993 (109) TRUCK ADT 5 %
(19) BYPASS, DETOUR LENGTH 19 KM

*************** GEOMETRIC DATA ***************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0.2 M RIGHT 0.2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 45.1
STATUS STRUCTURALLY DEFICIENT
HEALTH INDEX 85.1
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- NOT ON NHS 0
(26) FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY- NOT STRAHNET 0
(101) PARALLEL STRUCTURE- NONE EXISTS N
(102) DIRECTION OF TRAFFIC- 2 WAY 2
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- COUNTY HIGHWAY AGENCY 02
(22) OWNER- COUNTY HIGHWAY AGENCY 02
(37) HISTORICAL SIGNIFICANCE- ELIGIBLE 2

**************** CONDITION **************** CODE

(58) DECK 3
(59) SUPERSTRUCTURE 6
(60) SUBSTRUCTURE 5
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- OTHER OR UNKNOWN 0
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 40.8
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 24.5
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 5
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0000
(113) SCOUR CRITICAL BRIDGES U

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $336,000
(95) ROADWAY IMPROVEMENT COST $34,000
(96) TOTAL PROJECT COST $504,000
(97) YEAR OF IMPROVEMENT COST ESTIMATE 1999
(114) FUTURE ADT 1420
(115) YEAR OF FUTURE ADT 2015

*************** INSPECTIONS ***************

(90) INSPECTION DATE 04/05 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number : 23C0092
Facility Carried: STEVENSON BR RD
Location : SOL/YOL CO LINE
City :
Inspection Date : 03/27/2003

Bridge Inspection Report

Inspection Type

Routine	Group A	Underwater	Special	Other
X				

STRUCTURE NAME: PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened: N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC tied arches with RC (5) girder approach spans on 2-column piers and RC piles, seat abutments. All founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load: OTHER OR UNKNOWN
Inventory Rating: 24.5 metric tons Calculation Method: LOAD FACTOR
Operating Rating: 40.8 metric tons Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A Type 3S2 N/A Type 3-3 N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width: 7.1 m Net Width: 6.1 m No. of Lanes: 2
Rail Description: Concrete. Rail Code : 0000
Min. Vertical Clearance: 4.310

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION OF STRUCTURE

As noted in previous reports, the deck has large transverse deck cracks approximately 3 to 4 m on center which appear to correspond with the locations of the floor beams.

There are large transverse soffit cracks in Spans 1 and 3, full width. Some have rebar exposed with rust staining.

The minor cracks and spalling in the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The retaining wall, which protects the bank in Span 1, has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of the wall has not changed and remains at 300 mm as previously reported.

The girders have a few spalls with rebar exposed. Previous patches are starting to fall out.

The bridge rail has random spalls and delaminations with rebar exposed throughout.

Span 1, Girder 1-3, approximately 2.5 m from Bent 2, there are 3/4 depth, full width, severe size (20 mm wide) girder cracks with full thickness deck cracking with severe rust staining and exposed rebar. The deck cracks are patched with AC.

Span 4 exhibits similar cracking throughout all girders with cracking being slightly more severe on the RH side of the structure.

Stream cross section was not taken due to swift current in the creek at this time of inspection.

MISCELLANEOUS

This bridge is labeled Structurally Deficient in the NBI status as defined by the FHWA. A bridge is considered Structurally Deficient when its capacity is less than the standards determined by the FHWA. The formula for calculating structural deficiency uses the following condition ratings: Deck (Item 58), Superstructure (Item 59), Substructure (Item 60), Culvert (Item 62), Structural Evaluation (Item 67) and Waterway Adequacy (Item 71).

The Approach Roadway Alignment (Item 72) rating of this bridge is 3, resulting in structural deficiency. For further information refer to the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", Report No. FHWA-PD-96-001.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT INSPECTION RATINGS

F #	Elem No.	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4	St. 5
01	12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
01	110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	0
01	144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
01	155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
01	205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	
01	215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	0
01	331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	0
01	358	Deck Cracking	2	1	ea.	0	0	0	1	0
01	359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
01	360	Settlement	2	1	ea.	0	1	0	0	0
01	361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

RecDate: 05/02/2000
Action :
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget:
DistTarget:
EA:

Place 14' 2" mininum vertical clearance signs at both approaches.

RecDate: 09/07/1995
Action :
Work By: LOCAL AGENCY
Status : PROPOSED

EstCost:
StrTarget:
DistTarget:
EA:

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and protected. Large spalled areas to be patched. The spalls over the roadway (portal bracing) should not be patched.

Inspected By : Michael Nguyen

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
MICHAEL VAN NGUYEN
No. C 057507
Exp. 12/31/05
CIVIL
STATE OF CALIFORNIA

CC: Hydraulics: Charles Ineichen
Anthony Gugino: Ratings
Mark Palmer: Geotechnical Services

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1)	STATE NAME- CALIFORNIA	069
(8)	STRUCTURE NUMBER	**23C0092**
(5)	INVENTORY ROUTE(ON/UNDER)- ON	1400W8510
(2)	HIGHWAY AGENCY DISTRICT	04
(3)	COUNTY CODE 095 (4) PLACE CODE	00000
(6)	FEATURE INTERSECTED-	PUTAH CREEK
(7)	FACILITY CARRIED-	STEVENSON BR RD
(9)	LOCATION-	SOL/YOL CO LINE
(11)	MILEPOINT/KILOMETERPOINT	0
(12)	BASE HIGHWAY NETWORK- NOT ON NET	0
(13)	LRS INVENTORY ROUTE & SUBROUTE	
(16)	LATITUDE	38 DEG 32 MIN 13 SEC
(17)	LONGITUDE	121 DEG 51 MIN 03 SEC
(98)	BORDER BRIDGE STATE CODE % SHARE	%
(99)	BORDER BRIDGE STRUCTURE NUMBER	

******** STRUCTURE TYPE AND MATERIAL *********

(43)	STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT TYPE- ARCH - THRU	CODE 212
(44)	STRUCTURE TYPE APPR:MATERIAL- TYPE- TEE BEAM	CODE 200
(45)	NUMBER OF SPANS IN MAIN UNIT	2
(46)	NUMBER OF APPROACH SPANS	2
(107)	DECK STRUCTURE TYPE- CIP CONCRETE	CODE 1
(108)	WEARING SURFACE / PROTECTIVE SYSTEM:	
A)	TYPE OF WEARING SURFACE- CONCRETE	CODE 1
B)	TYPE OF MEMBRANE- NONE	CODE 0
C)	TYPE OF DECK PROTECTION- NONE	CODE 0

*************** AGE AND SERVICE ***************

(27)	YEAR BUILT	1923
(106)	YEAR RECONSTRUCTED	0000
(42)	TYPE OF SERVICE: ON- HIGHWAY	1
	UNDER- WATERWAY	5
(28)	LANES: ON STRUCTURE 02 UNDER STRUCTURE	
(29)	AVERAGE DAILY TRAFFIC	900
(30)	YEAR OF ADT 1993 (109) TRUCK ADT	5 %
(19)	BYPASS, DETOUR LENGTH	19 KM

*************** GEOMETRIC DATA ****************

(48)	LENGTH OF MAXIMUM SPAN	32.9M
(49)	STRUCTURE LENGTH	90.8M
(50)	CURB OR SIDEWALK: LEFT .2M RIGHT	.2M
(51)	BRIDGE ROADWAY WIDTH CURB TO CURB	6.1M
(52)	DECK WIDTH OUT TO OUT	7.1M
(32)	APPROACH ROADWAY WIDTH (W/SHOULDERS)	5.8M
(33)	BRIDGE MEDIAN- NO MEDIAN	0
(34)	SKEW 0 DEG (35) STRUCTURE FLARED	NO
(10)	INVENTORY ROUTE MIN VERT CLEAR	4.31M
(47)	INVENTORY ROUTE TOTAL HORIZ CLEAR	6.1M
(53)	MIN VERT CLEAR OVER BRIDGE RDWY	4.31M
(54)	MIN VERT UNDERCLEAR REF- NOT H/RR	0 M
(55)	MIN LAT UNDERCLEAR RT REF-	99.9M
(56)	MIN LAT UNDERCLEAR LT	0 M

*************** NAVIGATION DATA ***************

(38)	NAVIGATION CONTROL- NOT APPLICABLE	CODE N
(111)	PIER PROTECTION-	CODE
(39)	NAVIGATION VERTICAL CLEARANCE	0M
(116)	VERT-LIFT BRIDGE NAV MIN VERT CLEAR	M
(40)	NAVIGATION HORIZONTAL CLEARANCE	0M

****

SUFFICIENCY RATING =	45.1
STATUS	STRUCTURALLY DEFICIENT
HEALTH INDEX =	85.1
PAINT CONDITION INDEX =	N/A

************* CLASSIFICATION ************* CODE

(112)	NBIS BRIDGE LENGTH- YES	Y
(104)	HIGHWAY SYSTEM- NOT ON NHS	0
(26)	FUNCTIONAL CLASS- MAJOR COLLECTOR RURAL	07
(100)	DEFENSE HIGHWAY- NOT STRAHNET	0
(101)	PARALLEL STRUCTURE- NONE EXISTS	N
(102)	DIRECTION OF TRAFFIC- 2 WAY	2
(103)	TEMPORARY STRUCTURE-	
(105)	FED.LANDS HWY-	
(110)	DESIGNATED NATIONAL NETWORK - NOT ON NET	0
(20)	TOLL- ON FREE ROAD	3
(21)	MAINTAIN- COUNTY HIGHWAY AGENCY	02
(22)	OWNER- COUNTY HIGHWAY AGENCY	02
(37)	HISTORICAL SIGNIFICANCE- ELIGIBLE	2

**************** CONDITION **************** CODE

(58)	DECK	3
(59)	SUPERSTRUCTURE	6
(60)	SUBSTRUCTURE	5
(61)	CHANNEL & CHANNEL PROTECTION	6
(62)	CULVERTS	N

******** LOAD RATING AND POSTING ********* CODE

(31)	DESIGN LOAD- OTHER OR UNKNOWN	0
(63)	OPERATING RATING METHOD- LOAD FACTOR	1
(64)	OPERATING RATING-	40.8
(65)	INVENTORY RATING METHOD- LOAD FACTOR	1
(66)	INVENTORY RATING-	24.5
(70)	BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS	5
(41)	STRUCTURE OPEN, POSTED OR CLOSED-	A
	DESCRIPTION- OPEN, NO RESTRICTION	

**************** APPRAISAL **************** CODE

(67)	STRUCTURAL EVALUATION	5
(68)	DECK GEOMETRY	3
(69)	UNDERCLEARANCES, VERTICAL & HORIZONTAL	N
(71)	WATER ADEQUACY	7
(72)	APPROACH ROADWAY ALIGNMENT	3
(36)	TRAFFIC SAFETY FEATURES	0000
(113)	SCOUR CRITICAL BRIDGES	U

************ PROPOSED IMPROVEMENTS ************

(75)	TYPE OF WORK- DECK REHABILITATION	CODE 36
(76)	LENGTH OF STRUCTURE IMPROVEMENT	90.8M
(94)	BRIDGE IMPROVEMENT COST	$336,000
(95)	ROADWAY IMPROVEMENT COST	$34,000
(96)	TOTAL PROJECT COST	$504,000
(97)	YEAR OF IMPROVEMENT COST ESTIMATE	1999
(114)	FUTURE ADT	1420
(115)	YEAR OF FUTURE ADT	2015

***************** INSPECTIONS *****************

(90)	INSPECTION DATE 03/03 (91) FREQUENCY	24 MO
(92)	CRITICAL FEATURE INSPECTION:	(93) CFI DATE
A)	FRACTURE CRIT DETAIL- NO -1 MO	A)
B)	UNDERWATER INSP- NO -1 MO	B)
C)	OTHER SPECIAL INSP- NO -1 MO	C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number	:	**23C0092**
Facility Carried:		**STEVENSON BR RD**
Location	:	**SOL/YOL CO LINE**
City	:	
Inspection Date	:	**23-JAN-02**

Inspection Type

Routine	Group A	Underwater	Special	Other
X				

Bridge Inspection Report

Name : PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built :	1923	Skew (degrees):	0
Year Widened :	N/A	No. of Joints :	0
Length (m) :	90.8	No. of Hinges :	0

Description of Structure : RC tied arches with RC (5) girder approach spans on 2-column piers and RC piles, seat abutments. All founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load : OTHER OR UNKNOWN
Inventory Rating : 24.5 metric tons Calculation Method : LOAD FACTOR
Operating Rating : 40.8 metric tons Calculation Method : LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A english tons Type 3S2 N/A english tons Type 3-3 N/A english tons

DESCRIPTION ON STRUCTURE

Bridge width : 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width : 7.1 m Net Width : 6.10 m No. of Lanes : 2
Rail Description : Concrete. Rail Code : 0000
Min. Vertical Clearance : 4.310 m

DESCRIPTION UNDER STRUCTURE

Channel Description : Sand and gravel.

REVISIONS

ELI Element 12 - Concrete Deck - Bare - 560 sq m moved to condition state 2 from condition state 1 to conform with conditions observed in the field.

ELI Element 110 - Reinforced Conc Open Girder/Beam - 20 m moved to condition state 2 (total of 30 m) from condition state 1 and 10 m moved to condition state 3 (total of 20 m) from condition state 1 to conform with conditions observed in the field.

ELI Element 331 - Reinforced Conc Bridge Railing - 60 m moved to condition state 3 (total of 83 m) from condition state 2 to conform with conditions observed in the field.

ELI Element 358 - Deck Cracking - 1 ea moved to condition state 4 from condition state 3 to conform with conditions observed in the field.

ELI Element 359 - Soffit Cracking - 1 ea moved to condition state 5 from condition state 3 to conform with conditions observed in the field.

ELI Element 360 - Settlement - 1 ea added in condition state 2 as both abutments show signs of excessive settlement which is causing severe cracks in the girders of Span 1 and 4.

CONDITION OF STRUCTURE

As noted in previous reports, the deck has large transverse deck cracks approximately 3-4 m on center which appear to correspond with the locations of the floor beams.

There are large transverse soffit cracks in Spans 1 and 3, full width. Some have rebar exposed with rust staining.

The minor cracks and spalling in the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The retaining wall, which protects the bank in Span 1, has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of the wall has not changed and remains at 300 mm as previously reported.

The girders have a few spalls with rebar exposed. Previous patches are starting to fall out.

The bridge rail has random spalls and delaminations with rebar exposed throughout.

Span 1, Girder 1-3, approximately 2.5 m from Bent 2, there are 3/4 depth, full width, severe size (20 mm wide) girder cracks with full thickness deck cracking with severe rust staining and exposed rebar. The deck cracks are patched with AC.

Span 4 exhibits similar cracking throughout all girders with cracking being slightly more severe on the RH side of the structure.

This bridge is currently classified as Structurally Deficient due to the generally poor condition of the deck as well as the deteriorated condition of the substructure.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT LEVEL INSPECTION RATINGS

F#	Elem No.	Element Description	Env	Total Quantity	Units	St. 1	St. 2	St. 3	St. 4	St. 5
01	12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
01	110	Reinforced Conc Open Girder/Beam	2	122	m.	72	30	20	0	
01	144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
01	155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
01	205	Reinforced Conc Column or Pile Extension	2	6	ea.	4	2	0	0	
01	215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	
01	331	Reinforced Conc Bridge Railing	2	183	m.	0	100	83	0	
01	358	Deck Cracking	2	1	ea.	0	0	0	1	
01	359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	0	0	1
01	360	Settlement	2	1	ea.	0	1	0		
01	361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and protected. Large spalled areas to be patched. The spalls over the roadway (portal bracing) should not be patched.

Item#	Rec. Date	Work By	Work Id.	Prog. Method	Cost
1	07-SEP-1995	County Agency	40092X95250X		

Place 14' 2" mininum vertical clearance signs at both approaches.

Item#	Rec. Date	Work By	Work Id.	Prog. Method	Cost
2	02-MAY-2000	County Agency	40092X00123X		

Inspected By : Chuck Laughlin

Registered Civil Engineer

No.CE31605
Exp. 6-30-05
CIVIL

CC: Eric Bost Ct Local Programs

Printed on : 10-APR-2002 02:32:19 PM

Bridge No.: 23C0092 Location: SOL/YOL CO LINE Inspection Date: 23-JAN-02

STRUCTURE INVENTORY AND APPRAISAL REPORT

********************** IDENTIFICATION *****************

(1) STATE NAME - CALIFORNIA 069
(8) STRUCTURE NUMBER 23C0092
(5) INVENTORY ROUTE(ON/UNDER) - ON 1 40 0W8510
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED - PUTAH CREEK
(7) FACILITY CARRIED - STEVENSON BR RD
(9) LOCATION - SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK - NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 13 SEC
(17) LONGITUDE 121 DEG 51 MIN 03 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

*************** STRUCTURE TYPE AND MATERIAL ***********

(43) STRUCTURE TYPE MAIN: MATERIAL - CONCRETE CONT
TYPE - ARCH - THRU CODE 2 12
(44) STRUCTURE TYPE APPR: MATERIAL - CONCRETE CONT
TYPE - TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE - CONCRETE CODE 1
B) TYPE OF MEMBRANE - NONE CODE 0
C) TYPE OF DECK PROTECTION - NONE CODE 0

********************* AGE AND SERVICE *****************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON - HIGHWAY 1
UNDER - WATERWAY 5
(28) LANES: ON STRUCTURE 02 UNDER STRUCTURE
(29) AVERAGE DAILY TRAFFIC 900
(30) YEAR OF ADT 1998 (109) TRUCK ADT 5%
(19) BYPASS, DETOUR LENGTH 19 KM

******************* GEOMETRIC DATA *******************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT .2 M RIGHT .2 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.1 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN - NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.31 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.31 M
(54) MIN VERT UNDERCLEAR REF - NOT H/RR 0 M
(55) MIN LAT UNDERCLEAR RT REF - 99.9 M
(56) MIN LAT UNDERCLEAR LT 0 M

******************* NAVIGATION DATA ******************

(38) NAVIGATION CONTROL - NOT APPLICABLE CODE N
(111) PIER PROTECTION - CODE
(39) NAVIGATION VERTICAL CLEARANCE 0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0

****

SUFFICIENCY RATING = 37.0
STATUS = STRUCTURALLY DEFICIENT
HEALTH INDEX = 82.02

**************** CLASSIFICATION **************** CODE

(112) NBIS BRIDGE LENGTH - YES Y
(104) HIGHWAY SYSTEM - NOT ON NHS 0
(26) FUNCTIONAL CLASS - MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY - NOT STRAHNET 0
(101) PARALLEL STRUCTURE - NONE EXISTS N
(102) DIRECTION OF TRAFFIC - 2 WAY 2
(103) TEMPORARY STRUCTURE -
(105) FEDERAL LANDS HIGHWAY -
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL - ON FREE ROAD 3
(21) MAINTAIN - COUNTY HIGHWAY AGENCY 2
(22) OWNER - COUNTY HIGHWAY AGENCY 2
(37) HISTORICAL SIGNIFICANCE - ELIGIBLE 2

****************** CONDITION ****************** CODE

(58) DECK 3
(59) SUPERSTRUCTURE 6
(60) SUBSTRUCTURE 4
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

************ LOAD RATING AND POSTING *********** CODE

(31) DESIGN LOAD - OTHER OR UNKNOWN 0
(63) OPERATING RATING METHOD - LOAD FACTOR 1
(64) OPERATING RATING - 40.8
(65) INVENTORY RATING METHOD - LOAD FACTOR 1
(66) INVENTORY RATING - 24.5
(70) BRIDGE POSTING - Equal to or above legal loads 5
(41) STRUCTURE OPEN, POSTED OR CLOSED - A
DESCRIPTION - OPEN, NO RESTRICTION

****************** APPRAISAL ****************** CODE

(67) STRUCTURAL EVALUATION 4
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 7
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 0000
(113) SCOUR CRITICAL BRIDGES U

**************** PROPOSED IMPROVEMENTS ***************

(75) TYPE OF WORK - DECK REHABILITATION CODE 36
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $336,000
(95) ROADWAY IMPROVEMENT COST $34,000
(96) TOTAL PROJECT COST $504,000
(97) YEAR OF IMPROVEMENT COST ESTIMATE 1999
(114) FUTURE ADT 1420
(115) YEAR OF FUTURE ADT 2015

******************* INSPECTIONS *******************

(90) INSPECTION DATE 01/02 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL - NO -1 MO A)
B) UNDERWATER INSP - NO -1 MO B)
C) OTHER SPECIAL INSP - NO -1 MO C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Bridge Number :	**23C0092**
Facility Carried:	**STEVENSON BR RD**
Location :	**SOL/YOL CO LINE**
City :	
Inspection Date :	**02-MAY-00**

Inspection Type

Routine	Group A	Underwater	Special	Other
X				

Bridge Inspection Report

Name : PUTAH CREEK

CONSTRUCTION INFORMATION

Year Built : 1923
Year Widened : N/A
Length (m) : 90.8

Skew (degrees): 0
No. of Joints : 0
No. of Hinges : 0

Description of Structure : RC tied arches with RC (5) girder approach spans on 2-column piers and RC piles, seat abutments. All founded on spread footings.

Span Configuration : 12.2 m, 2 @ 32.9 m, 12.2 m

LOAD CAPACITY AND RATINGS

Design Live Load : OTHER OR UNKNOWN
Inventory Rating : 24.5 metric tons — Calculation Method : LOAD FACTOR
Operating Rating : 40.8 metric tons — Calculation Method : LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A english tons Type 3S2 N/A english tons Type 3-3 N/A english tons

DESCRIPTION ON STRUCTURE

Bridge width : 0.3 m r, 0.2 m cu, 6.1 m, 0.2 m cu, 0.3 m r
Total Width : 7.1 m — Net Width : 6.10 m — No. of Lanes : 2
Rail Description : Concrete. — Rail Code : 0000
Min. Vertical Clearance : 4.310 m

DESCRIPTION UNDER STRUCTURE

Channel Description : Sand and gravel.

CONDITION OF STRUCTURE

As noted in previous reports, the deck has large transverse deck cracks approximately 3-4 m on centers, and remains unchanged.

The deck was chained and there were no delaminations noted.

There are large transverse soffit cracks in Spans 1 and 3, full width. Some have rebar exposed with rust staining.

The minor cracks and spalling in the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The retaining wall, which protects the bank in Span 1, has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of the wall has not changed and remains at 300 mm as previously reported.

The girders have a few spalls with rebar exposed. Previous patches are starting to fall out.

The bridge rail has random spalls and delaminations with rebar exposed throughout.

The minimum vertical clearance was measured to be 14'2".

SCOUR

The footing is exposed 1 m at Pier 3, no undermining.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

Bridge No.: 23C0092 **Location: SOL/YOL CO LINE** **Inspection Date: 02-MAY-00**

ELEMENT LEVEL INSPECTION RATINGS

F#	Elem No.	Element Description	Env	Total Quantity	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4	St. 5
01	12	Concrete Deck - Bare	2	560	sq.m.	560	0	0	0	0
01	110	Reinforced Conc Open Girder/Beam	2	122	m.	102	10	10	0	
01	144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
01	155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
01	205	Reinforced Conc Column or Pile Extension	2	6	ea.	4	2	0	0	
01	215	Reinforced Conc Abutment	2	16	m.	8	8	0	0	
01	331	Reinforced Conc Bridge Railing	2	183	m.	0	160	23	0	
01	358	Deck Cracking	2	1	ea.	0	0	1	0	
01	359	Soffit of Concrete Deck or Slab	2	1	ea.	0	0	1	0	0
01	361	Scour	3	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and protected. Large spalled areas to be patched. The spalls over the roadway (portal bracing) should not be patched.

Item#	Rec. Date	Work By	Work Id.	Prog. Method	Cost
1	07-SEP-1995	County Agency	40092X95250X		

Place 14' 2" miminum vertical clearance signs at both approaches.

Item#	Rec. Date	Work By	Work Id.	Prog. Method	Cost
2	02-MAY-2000	County Agency	40092X00123X		

Inspected By : Patrick Piacentini Pat Piacentini

Richard C. Dills

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
RICHARD C. DILLS
No. S 4069
Exp. 3-31-2004
STRUCTURAL
STATE OF CALIFORNIA

Bridge No.: 23C0092 Location: SOL/YOL CO LINE Inspection Date: 02-MAY-00

STRUCTURE INVENTORY AND APPRAISAL REPORT

********************* IDENTIFICATION ****************

(1)	STATE NAME - CALIFORNIA	069
(8)	STRUCTURE NUMBER	23C0092
(5)	INVENTORY ROUTE(ON/UNDER) - ON	1 40 0W8510
(2)	HIGHWAY AGENCY DISTRICT	04
(3)	COUNTY CODE 095 (4) PLACE CODE	00000
(6)	FEATURE INTERSECTED - PUTAH CREEK	
(7)	FACILITY CARRIED - STEVENSON BR RD	
(9)	LOCATION - SOL/YOL CO LINE	
(11)	MILEPOINT/KILOMETERPOINT	0
(12)	BASE HIGHWAY NETWORK - NOT ON NET	0
(13)	LRS INVENTORY ROUTE & SUBROUTE	
(16)	LATITUDE	38 DEG 32 MIN 24 SEC
(17)	LONGITUDE	121 DEG 51 MIN 06 SEC
(98)	BORDER BRIDGE STATE CODE % SHARE	%
(99)	BORDER BRIDGE STRUCTURE NUMBER	

*************** STRUCTURE TYPE AND MATERIAL **********

(43)	STRUCTURE TYPE MAIN: MATERIAL - CONCRETE CONT	
	TYPE - ARCH - THRU	CODE 2 12
(44)	STRUCTURE TYPE APPR: MATERIAL - CONCRETE CONT	
	TYPE - TEE BEAM	CODE 204
(45)	NUMBER OF SPANS IN MAIN UNIT	2
(46)	NUMBER OF APPROACH SPANS	2
(107)	DECK STRUCTURE TYPE CIP CONCRETE	CODE 1
(108)	WEARING SURFACE / PROTECTIVE SYSTEM:	
	A) TYPE OF WEARING SURFACE - CONCRETE	CODE 1
	B) TYPE OF MEMBRANE - NONE	CODE 0
	C) TYPE OF DECK PROTECTION - NONE	CODE 0

******************** AGE AND SERVICE ****************

(27)	YEAR BUILT	1923
(106)	YEAR RECONSTRUCTED	0000
(42)	TYPE OF SERVICE: ON - HIGHWAY	1
	UNDER - WATERWAY	5
(28)	LANES: ON STRUCTURE 02 UNDER STRUCTURE	
(29)	AVERAGE DAILY TRAFFIC	900
(30)	YEAR OF ADT 1998 (109) TRUCK ADT	5%
(19)	BYPASS, DETOUR LENGTH	19 KM

****************** GEOMETRIC DATA ******************

(48)	LENGTH OF MAXIMUM SPAN	32.9 M
(49)	STRUCTURE LENGTH	90.8 M
(50)	CURB OR SIDEWALK: LEFT .2 M RIGHT	.2 M
(51)	BRIDGE ROADWAY WIDTH CURB TO CURB	6.1 M
(52)	DECK WIDTH OUT TO OUT	7.1 M
(32)	APPROACH ROADWAY WIDTH (W/SHOULDERS)	5.8 M
(33)	BRIDGE MEDIAN - NO MEDIAN	0
(34)	SKEW 0 DEG (35) STRUCTURE FLARED	NO
(10)	INVENTORY ROUTE MIN VERT CLEAR	4.31 M
(47)	INVENTORY ROUTE TOTAL HORIZ CLEAR	6.1 M
(53)	MIN VERT CLEAR OVER BRIDGE RDWY	4.31 M
(54)	MIN VERT UNDERCLEAR REF - NOT H/RR	0 M
(55)	MIN LAT UNDERCLEAR RT REF -	99.9 M
(56)	MIN LAT UNDERCLEAR LT	0 M

****************** NAVIGATION DATA *****************

(38)	NAVIGATION CONTROL - NOT APPLICABLE	CODE N
(111)	PIER PROTECTION -	CODE
(39)	NAVIGATION VERTICAL CLEARANCE	0 M
(116)	VERT-LIFT BRIDGE NAV MIN VERT CLEAR	M
(40)	NAVIGATION HORIZONTAL CLEARANCE	0

****

SUFFICIENCY RATING = 46.1

STATUS = STRUCTURALLY DEFICIENT

HEALTH INDEX = 87.3

*************** CLASSIFICATION *************** CODE

(112)	NBIS BRIDGE LENGTH - YES	Y
(104)	HIGHWAY SYSTEM - NOT ON NHS	0
(26)	FUNCTIONAL CLASS - MAJOR COLLECTOR RURAL	07
(100)	DEFENSE HIGHWAY - NOT STRAHNET	0
(101)	PARALLEL STRUCTURE - NONE EXISTS	N
(102)	DIRECTION OF TRAFFIC - 2 WAY	2
(103)	TEMPORARY STRUCTURE -	
(105)	FEDERAL LANDS HIGHWAY -	
(110)	DESIGNATED NATIONAL NETWORK - NOT ON NET	0
(20)	TOLL - ON FREE ROAD	3
(21)	MAINTAIN - COUNTY HIGHWAY AGENCY	2
(22)	OWNER - COUNTY HIGHWAY AGENCY	2
(37)	HISTORICAL SIGNIFICANCE - ELIGIBLE	2

***************** CONDITION ****************** CODE

(58)	DECK	4
(59)	SUPERSTRUCTURE	6
(60)	SUBSTRUCTURE	5
(61)	CHANNEL & CHANNEL PROTECTION	6
(62)	CULVERTS	N

*********** LOAD RATING AND POSTING *********** CODE

(31)	DESIGN LOAD - OTHER OR UNKNOWN	0
(63)	OPERATING RATING METHOD - LOAD FACTOR	1
(64)	OPERATING RATING -	40.8
(65)	INVENTORY RATING METHOD - LOAD FACTOR	1
(66)	INVENTORY RATING -	24.5
(70)	BRIDGE POSTING - Equal to or above legal loads	5
(41)	STRUCTURE OPEN, POSTED OR CLOSED -	A
	DESCRIPTION - OPEN, NO RESTRICTION	

***************** APPRAISAL ****************** CODE

(67)	STRUCTURAL EVALUATION	5
(68)	DECK GEOMETRY	3
(69)	UNDERCLEARANCES, VERTICAL & HORIZONTAL	N
(71)	WATER ADEQUACY	7
(72)	APPROACH ROADWAY ALIGNMENT	3
(36)	TRAFFIC SAFETY FEATURES	0000
(113)	SCOUR CRITICAL BRIDGES	U

*************** PROPOSED IMPROVEMENTS **************

(75)	TYPE OF WORK - DECK REHABILITATION	CODE 36
(76)	LENGTH OF STRUCTURE IMPROVEMENT	90.8 M
(94)	BRIDGE IMPROVEMENT COST	$336,000
(95)	ROADWAY IMPROVEMENT COST	$34,000
(96)	TOTAL PROJECT COST	$504,000
(97)	YEAR OF IMPROVEMENT COST ESTIMATE	1999
(114)	FUTURE ADT	1420
(115)	YEAR OF FUTURE ADT	2015

******************* INSPECTIONS ********************

(90) INSPECTION DATE 05/00 (91) FREQUENCY 24 MO

(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE

A) FRACTURE CRIT DETAIL -	NO	-1 MO	A)
B) UNDERWATER INSP -	NO	-1 MO	B)
C) OTHER SPECIAL INSP -	NO	-1 MO	C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigation

Caltrans

Bridge Inspection Report

Bridge Number : **23C0092**
Location : **SOL/YOL CO LINE**
Inspection Date : **30-SEP-97**

Inspection Type

Routine	Group A	Underwater	Special	Other
X				

Name : PUTAH CREEK

CONDITION OF STRUCTURE

Deck cracking in all spans has been documented in earlier reports and remains unchanged.

Minor cracks and spalling of the arch members have also been documented in past reports and further deterioration has not advanced significantly to warrant any analysis.

The retaining wall, which protects the bank in Span 1, has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of the wall has not changed and remains at 300 mm.

Pier 3, which is partially covered with berry vines, was found to have no undermining of the footing, but is exposed approximately 0.5 to 1.0 meters. No scour was present.

There is still no vertical clearance signs posted at this site. At one time there were signs that read: (14'-3"), at the portals.

This structure remains in fair condition.

SIGNS

There are signs in place at both approaches that read "NARROW BRIDGE".

ELEMENT LEVEL INSPECTION RATINGS

F#	Elem No.	Element Description	Env	Total Quantity	Units	St. 1	St. 2	St. 3	St. 4	St. 5
01	12	Concrete Deck - Bare	2	560	sq.m.	0	560	0	0	0
01	110	Reinforced Conc Open Girder/Beam	2	122	m.	122	0	0	0	0
01	144	Reinforced Conc Arch	2	132	m.	66	33	33	0	0
01	155	Reinforced Conc Floor Beam	2	180	m.	180	0	0	0	0
01	205	Reinforced Conc Column or Pile Extension	2	6	ea.	6	0	0	0	0
01	215	Reinforced Conc Abutment	2	15	m.	15	0	0	0	0
01	331	Reinforced Concrete Bridge Railing	2	183	m.	0	183	0	0	0
01	358	Deck Cracking	2	1	ea.	0	0	1	0	0
01	361	Scour	3	1	ea.	0	1	0	0	0

(Quantity in each Condition State: St. 1 – St. 5)

WORK RECOMMENDATIONS

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and protected. Large spalled areas to be patched. The spalls over the roadway (portal bracing) should not be patched.

Reco. Date	Work By	Work Id.	Prog. Method	Cost
07-SEP-1995	County Agency	40092X95249X		

Remeasure and replace vertical clearance signs on portals.

Reco. Date	Work By	Work Id.	Prog. Method	Cost
07-SEP-1995	County Agency	40092X95249X		

Inspected By : Paul Q. Lukkarila

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
RICHARD C. DILLS
NO. S 4069
EXP. 3-31-2000
STRUCTURAL
STATE OF CALIFORNIA

Printed on : 12-APR-1998 03:19:35 PM

Bridge No.: 23C0092 Location: SOL/YOL CO LINE Inspection Date: 30-SEP-97

STRUCTURE INVENTORY AND APPRAISAL REPORT

********************** IDENTIFICATION ****************

(1) STATE NAME - CALIFORNIA 069
(8) STRUCTURE NUMBER 23C0092
(5) INVENTORY ROUTE(ON/UNDER) - ON 1 40 0W8510
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 095 (4) PLACE CODE 00000
(6) FEATURE INTERSECTED - PUTAH CREEK
(7) FACILITY CARRIED - STEVENSON BR RD
(9) LOCATION - SOL/YOL CO LINE
(11) MILEPOINT/KILOMETERPOINT 0
(12) BASE HIGHWAY NETWORK - NOT ON NET 0
(13) LRS INVENTORY ROUTE & SUBROUTE
(16) LATITUDE 38 DEG 32 MIN 24 SEC
(17) LONGITUDE 121 DEG 51 MIN 06 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

*************** STRUCTURE TYPE AND MATERIAL **********

(43) STRUCTURE TYPE MAIN: MATERIAL - CONCRETE CONT
TYPE - ARCH - THRU CODE 2 12
(44) STRUCTURE TYPE APPR: MATERIAL - CONCRETE CONT
TYPE - TEE BEAM CODE 204
(45) NUMBER OF SPANS IN MAIN UNIT 2
(46) NUMBER OF APPROACH SPANS 2
(107) DECK STRUCTURE TYPE CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE - BITUMINOUS CODE 6
B) TYPE OF MEMBRANE - NONE CODE 0
C) TYPE OF DECK PROTECTION - NONE CODE 0

********************* AGE AND SERVICE ****************

(27) YEAR BUILT 1923
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON - HIGHWAY 1
UNDER - WATERWAY 5
(28) LANES: ON STRUCTURE 02 UNDER STRUCTURE
(29) AVERAGE DAILY TRAFFIC 900
(30) YEAR OF ADT 1993 (109) TRUCK ADT 5%
(19) BYPASS, DETOUR LENGTH 19 KM

******************* GEOMETRIC DATA ******************

(48) LENGTH OF MAXIMUM SPAN 32.9 M
(49) STRUCTURE LENGTH 90.8 M
(50) CURB OR SIDEWALK: LEFT 0 M RIGHT 0 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 6.1 M
(52) DECK WIDTH OUT TO OUT 7.4 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 5.8 M
(33) BRIDGE MEDIAN - NO MEDIAN 0
(34) SKEW 0 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 4.34 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 6.1 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 4.34 M
(54) MIN VERT UNDERCLEAR REF - NOT H/RR 0 M
(55) MIN LAT UNDERCLEAR RT REF - NOT H/RR 99.9 M
(56) MIN LAT UNDERCLEAR LT 0 M

******************* NAVIGATION DATA *****************

(38) NAVIGATION CONTROL - NO CONTROL CODE 0
(111) PIER PROTECTION - CODE
(39) NAVIGATION VERTICAL CLEARANCE 0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0

****

SUFFICIENCY RATING =58.5
STATUS = STRUCTURALLY DEFECIENT

**************** CLASSIFICATION **************** CODE

(112) NBIS BRIDGE LENGTH - YES Y
(104) HIGHWAY SYSTEM - NOT ON NHS 0
(26) FUNCTIONAL CLASS - MAJOR COLLECTOR RURAL 07
(100) DEFENSE HIGHWAY - NOT STRAHNET 0
(101) PARALLEL STRUCTURE - NONE EXISTS N
(102) DIRECTION OF TRAFFIC - 2 WAY 2
(103) TEMPORARY STRUCTURE -
(105) FEDERAL LANDS HIGHWAY -
(110) DESIGNATED NATIONAL NETWORK -
(20) TOLL - ON FREE ROAD 3
(21) MAINTAIN - COUNTY HIGHWAY AGENCY 2
(22) OWNER - COUNTY HIGHWAY AGENCY 2
(37) HISTORICAL SIGNIFICANCE - ELIGIBLE 2

****************** CONDITION ****************** CODE

(58) DECK 4
(59) SUPERSTRUCTURE 6
(60) SUBSTRCTURE 6
(61) CHANNEL & CHANNEL PROTECTION 6
(62) CULVERTS N

************ LOAD RATING AND POSTING *********** CODE

(31) DESIGN LOAD - OTHER OR UNKNOWN 0
(63) OPERATING RATING METHOD - LOAD FACTOR 1
(64) OPERATING RATING - 40.8
(65) INVENTORY RATING METHOD - LOAD FACTOR 1
(66) INVENTORY RATING - 24.5
(70) BRIDGE POSTING - NO POSTING REQUIRED 5
(41) STRUCTURE OPEN, POSTED OR CLOSED - A
DESCRIPTION - OPEN, NO RESTRICTION

****************** APPRAISAL ****************** CODE

(67) STRUCTURAL EVALUATION 6
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 8
(72) APPROACH ROADWAY ALIGNMENT 3
(36) TRAFFIC SAFETY FEATURES 1000
(113) SCOUR CRITICAL BRIDGES U

**************** PROPOSED IMPROVEMENTS **************

(75) TYPE OF WORK - SUP/SUB REHAB CODE 35
(76) LENGTH OF STRUCTURE IMPROVEMENT 90.8 M
(94) BRIDGE IMPROVEMENT COST $446,827
(95) ROADWAY IMPROVEMENT COST $44,683
(96) TOTAL PROJECT COST $670,241
(97) YEAR OF IMPROVEMENT COST ESTIMATE 1998
(114) FUTURE ADT 420
(115) YEAR OF FUTURE ADT 2010

******************** INSPECTIONS ********************

(90) INSPECTION DATE 09/97 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL - NO -1 MO A)
B) UNDERWATER INSP - NO -1 MO B)
C) OTHER SPECIAL INSP - NO -1 MO C)

Printed on : 12-APR-1998 03:19:35 PM

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Bridge No. 23C-0092

SUPPLEMENTARY BRIDGE REPORT
DS-M19(REV.1-90)

Location 4/3-Sol/Yol-FAS W851
Dist.,Co.,Rte.,PM,City

Date of Investigation 9/7/95

Name PUTAH CREEK (Stevenson Bridge Road)

RATINGS:
71 **Waterway Adequacy** 8   61 **Channel & Channel Protection** 6   72 **Approach Rdwy Align.** 3

TYPE OF INVESTIGATION/REPORT

Biennial	X	**Group A**		**Other**	
Damage		**Underwater**		**Office**	

WORK NOT DONE
Work recommended in the previous biennial bridge report has not been done.

CONDITION OF STRUCTURE
Deck cracking in all spans has been documented in earlier reports and is unchanged.

Minor cracks and spalling of the arch members have been documented in past reports and are also unchanged. Many of the spalls are still in place and the reinforcement cannot be examined.

The retaining wall which protects the bank in Span 1 has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of wall is about 300 mm. No signs of recent movement exist, however the wall is considered marginally stable and could rotate further or overturn if subjected to hydrostatic pressure following a period of heavy run-off.

The bank is eroded in Span 1 from pedestrian traffic and natural causes. The footing of Abutment 1 has apparently been underpinned twice. The top of the original footing is 2 m - 3 m above the present ground line at the downstream end. No undermining exists at this time.

Pier 3, which is exposed presently, is covered with berry vines, and the extent of exposure and/or undermining cannot be determined. Several years ago the piles (timber) were exposed at this support due to scour.

No vertical clearance signs exist at the site. At one time there were signs which read: (14'-3"), at the portals.

The structure remains in fair overall condition.

WORK RECOMMENDED
Clear brush and vines around Bent 3 for inspection. Notify undersigned when this work is to be done so that the inspection can be completed.

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and painted with epoxy. Large spalled areas can be patched. Spalls over the roadway, (portal bracing etc.) should not be patched.

Replace vertical clearance signs on portals.

BRIDGE NO. 23C-0092	
SHEET 2	DATE 9-7-95

<u>PONTIS INSPECTION</u>
A PONTIS inspection form for this investigation is attached.

William R. Baker
Registered Civil Engineer

WRB/pfa

REGISTERED PROFESSIONAL ENGINEER
WILLIAM R. BAKER
No 16500
Exp. 6/30/97
CIVIL
STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

SUPPLEMENTARY BRIDGE REPORT
DS-M19(REV.1-90)

Bridge No. 23C-0092

Location 10-Sol/Yol-FAS W851
Dist.,Co.,Rte.,PM,City

Date of Investigation 9-13-93

Name PUTAH CREEK (Stevenson Bridge Road)

RATINGS:
71 **Waterway Adequacy** 8 61 **Channel & Channel Protection** 6 72 **Approach Rdwy Align.** 3

TYPE OF INVESTIGATION/REPORT

Biennial	X	**Group A**		**Other**	
Damage		**Underwater**		**Office**	

WORK DONE
South approach pavement has been leveled at the end of bridge.

CONDITION OF STRUCTURE
Deck cracking in all spans has been documented in earlier reports and is unchanged.

Minor cracks and spalling of the arch members have been documented in past reports and are also unchanged. Many of the spalls are still in place and the reinforcement cannot be examined.

The retaining wall which protects the bank in Span 1 has, during the life of the structure, rotated away from the column at Bent 2. The offset at the top of wall is about 1 foot. No signs of recent movement exist, however the wall is considered marginally stable and could rotate further or overturn if subjected to hydrostatic pressure following a period of heavy run-off.

The bank is eroded in Span 1 from pedestrian traffic and natural causes. The footing of Abutment 1 has apparently been underpinned twice. The top of the original footing is 6'-8' above the present ground line at the downstream end. No undermining exists at this time.

Pier 3, which is exposed presently, is covered with berry vines, and the extent of exposure and/or undermining cannot be determined. Several years ago the piles (timber) were exposed at this support due to scour.

No vertical clearance signs exist at the site. At one time there were signs which read 14'-3" at the portals.

Numerous small stones have been bonded to the face of Pier 2. (rock climbers.)

WORK RECOMMENDED
Clear brush and vines around Bent 3 for inspection. MSCL

Remove remaining loose concrete from fractured areas on arch members. Corroded reinforcement should be blast cleaned and painted with epoxy. Large spalled areas can be patched. Spalls over the roadway, (portal bracing etc.) should not be patched. SUPM

Consider a tie back system to stabilize the Span 1 retaining wall. MSCH

Remove rock climber "steps" from pier face. MSCL

Replace vertical clearance signs on portals.

BRIDGE NO. 23C-0092	
SHEET 2	DATE 9-13-93

PONTIS INSPECTION
A PONTIS inspection form for this investigation is attached.

REGISTERED PROFESSIONAL ENGINEER
WILLIAM R. BAKER
NO. 16500
EXP. 06/30/97
CIVIL
STATE OF CALIFORNIA

William R. Baker
Registered Civil Engineer

WRB/sdy

cc: W. Lindsey-Hydraulics
Yolo County (2)

CHANNEL CROSS SECTION

BR. NO. 23C-92 NAME PUTAH CREEK LOCATION 10-Sol/Yol-FAS W851

PROFILE US(W) SIDE MEASURED FROM: top rail DATE 9/13/93

FROM	HORIZONTAL	VERTICAL	COMMENT
BB(S)	5	17+	face footing block @ S. Abut.
	34	26	base ret. wall (back)
	34+	16+	top " "
	35+	43	toe " " (front)
	41	43	Bt. 2
Bt. 2	68	53	ews } water < 1' deep
	80	53	" }
	108	47	Bt. 3
Bt. 3	55	43	-
	75	35	-
	108	24	Bt. 4
Bt. 4	38	17	face abut. (N)

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

SUPPLEMENTARY BRIDGE REPORT
DS-M19(REV.1-90)

Bridge No. 23C-0092
Location 10-Sol/Yol-FAS W851
Dist.,Co.,Rte.,PM,City
Date of Investigation 5-25-90

Name PUTAH CREEK (Stevenson Bridge Road)

RATINGS:

58 Deck 5 59 Superstructure 6 60 Substructure 5 71 Waterway Adequacy 8
61 Channel & Channel Protection 6 62 Culvert N 72 Approach Rdwy Align. 3

CODES:

21 Custodian 02 22 Owner 02 26 Functional Classification: Deck 07 Under
41 Str Open, Posted or Closed A 107 Deck Type 1 108 Wearing Surface/Prot Sys 600
Max Col/Pier Ht. Over 30' 111 Pier/Abut. Prot.
55 Min Lat Underclr on Rt. NA 54 Min Vert Underclr NA 112 NBIS Bridge Length Y

DATA:

51 Bridge Width (NET) 20.0' 109 Average Daily Trucks (% of ADT): Deck 05 Under
114 Future ADT: Deck 420 Under 115 Yr. of Future ADT: Deck 2010 Under
Number of Intermediate Joints: @ Hinges 0 @ Bents 2

TYPE OF INVESTIGATION/REPORT

Biennial X Category A Other
Damage Underwater Office

WORK NOT DONE
The large spall with exposed rebar on the south horizontal transverse portal member has not been patched. SUPL

CONDITION OF STRUCTURE
The south approach is up to 2 1/2" low in the northbound lane.

The present A.C. dike is proving inadequate to divert roadway runoff from Abutment 1, right.

There are now heavy to very heavy transverse cracks with edge spalling, at varied intervals, over the entire deck.

Rocks of various sizes have been cemented to the web wall and left column of Bent 2 to allow rock climbers to practice scaling vertical faces. Since this is an attractive nuisance, it is recommended that they be removed.

Graffiti covers the PCC rails and truss members.

There are car parts on the east side of Pier 2; and there are car parts, tires, signs, drift, and timber under Span 3.

The bridge is in fair condition.

BRIDGE NO. 23C-0092	
SHEET 2	DATE 5-25-90

WORK RECOMMENDED

1. Do the Work Not Done.

2. Level the south approach.

3. Build up the A.C. dike for 20' L.F. min. on the right side of road starting at Abutment 1.

4. Remove the rocks which are cemented to the web wall and left column of Bent 2. SUBL

5. Cover the graffiti on the PCC rails and truss members.

6. Remove the car parts from Pier 2 and the car parts, tires, signs, drift, and timber from under Span 3.

Chris B. Campbell

Chris B. Campbell
Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
CHRIS B. CAMPBELL
No. 37945
Exp. 3/31/93
CIVIL
STATE OF CALIFORNIA

CBC/ms-19290

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
SUPPLEMENTARY BRIDGE REPORT
DS-M19 (REV 7/87)

Bridge No. 23C-92

Location 10-Sol/Yol-FAS W851
Dist-Co-Rte-PM-City

Date of Investigation April 26, 1988

Name PUTAH CREEK (Stevenson Bridge Rd. @ Sol.-Yolo Co. Line)

CONDITION RATING: **APPRAISAL RATING:**

Deck 4 Superstructure (5) Substr. & Pipes 5 Overall 3

Channel & Channel Protection 5 Retaining Walls 5

Widenable? Yes ___ No X Conditional ___

Action Required By County: Yes X No ___

WORK NOT DONE:

The large spall with exposed rebar on the south horizontal transverse portal member has not been patched.

SUPL

CONDITION OF STRUCTURE:

Pier 3 footing is protected with a heavy blanket of rocks.

The condition of this structure has not changed significantly from the generally fair condition noted previously.

WORK RECOMMENDED:

Clean and patch the above-mentioned spall.

Vernon A. Banks
V. A. Banks
VAB/nlc

REGISTERED PROFESSIONAL ENGINEER
VERNON A. BANKS
No. C 25396
Exp. 12-31-89
CIVIL
STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

SIGNATURE REPORT

DS-M59 (REV. 10/76)

Bridge No. 23C-92

Name PUTAH CREEK (Stevensons Bridge)

10-Sol/Yol-FAS W851
Dist - Co - Rte - PM - City

On the date shown, the engineer investigated the structure and found no significant changes.

Date	Signature	Prev.Rec.Work Done	Prev.Rec.Work Not Done	Remarks
5-8-84	Vernon Banks			Rocks placed around Ps. GCNR
4-24-86	Vernon Banks			NCNR

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
SUPPLEMENTARY BRIDGE REPORT
DS-M19 (REV. 2/75)

Bridge No. 23C-92

Location 10-Sol/Yol-FAS/W851
Dist - Co - Rte - PM - City

Date of Investigation April 6, 1982

Name PUTAH CREEK (Stevenson Bridge Road)

CONDITION RATING: APPRAISAL RATING:

Deck 4 Superstructure 6 Substr. & Pipes 5 Overall 3

Channel & Channel Protection 5 Retaining Walls 5

Widenable? Yes ☐ No ☒ Conditional ☐

Action Required by District: Yes ☒ No ☐

PREVIOUS INVESTIGATION April 25, 1975

AVERAGE DAILY TRAFFIC 358 - 1973

BYPASS DETOUR LENGTH 12 miles

SEISMIC RETROFIT Need not be considered

RATINGS: Inventory HS15, Operating HS25, Permit PPPPP.

CONDITION OF STRUCTURE

In the 2nd truss span (from the Solano side) the 1st concrete portal bracing sustained damage from a recent overheight load. The concrete spall is 2' long x 6" x 6" with exposed rebar.

Much embankment has been washed away in front of Abutment 1. There is no undermining of the abutment footing but there is some at the right wingwall.

The water was too high to check the previously undermined bent 3 footing. This footing had 3' of exposed timber piles at the previous inspection.

LOAD CAPACITY

This structure calculates to be able to sustain all combinations of Legal Loads and the State's largest Permit Load.

This capacity is applicable for only as long as this structure remains in essentially the same condition as it was in during this investigation.

DS-M18 (REV. 6/75)

BRIDGE NO.	23C-92
SHEET: Two	DATE: April 6, 1982

WORK RECOMMENDED

Patch spall. At low water, treat exposed timber piles to help prevent decay.

Vernon A. Banks

Vernon A. Banks

VAB/sr

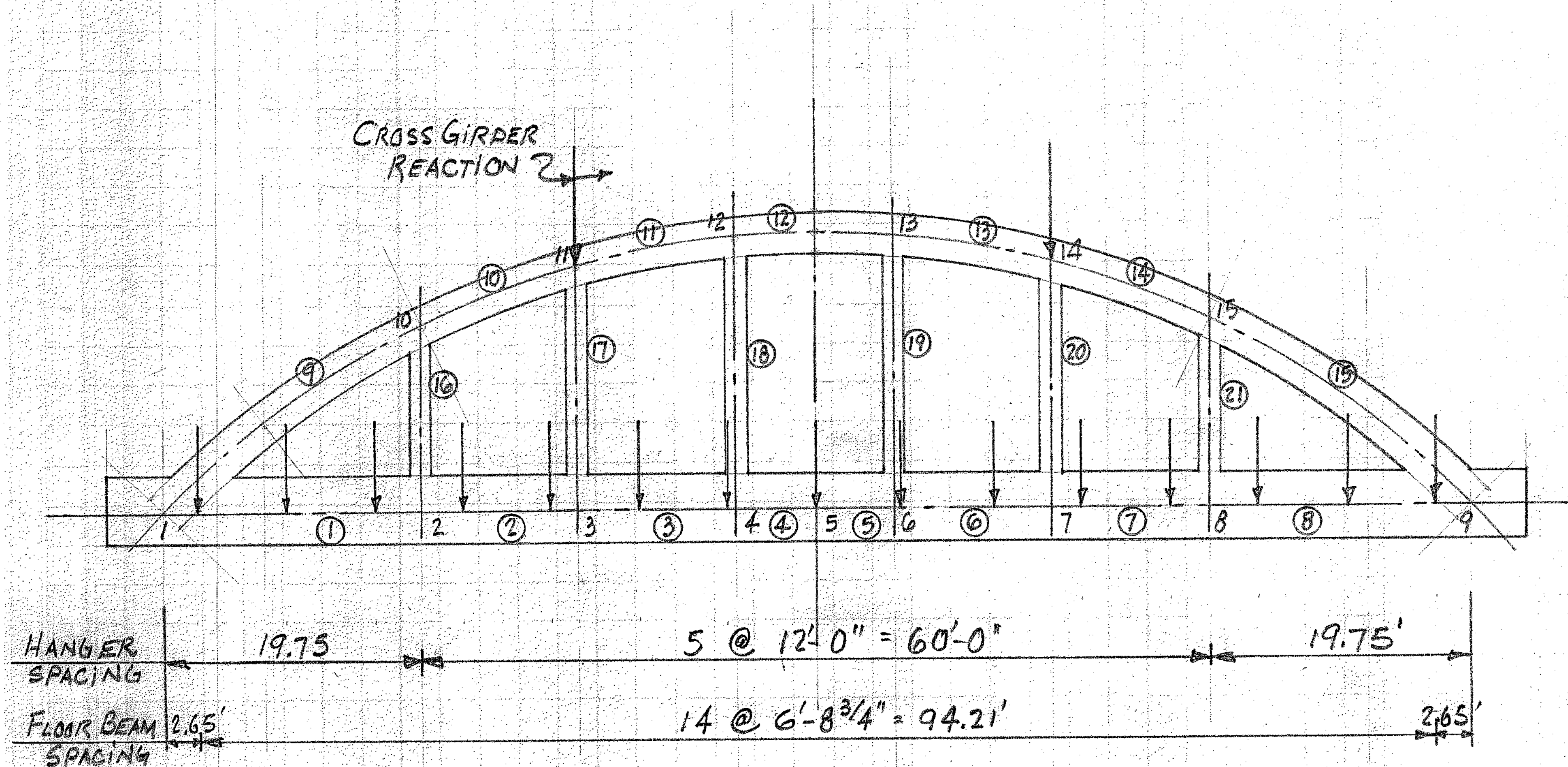
REGISTERED PROFESSIONAL ENGINEER
VERNON A. BANKS
No. C 25396
CIVIL
STATE OF CALIFORNIA

BY T.T. DATE 7/21/78 DS-D16 REV. 3-75

JOB PUTAH CREEK BR. (23C-92)

SUBJECT ARCH RATING

MODEL OF STRUCTURE:

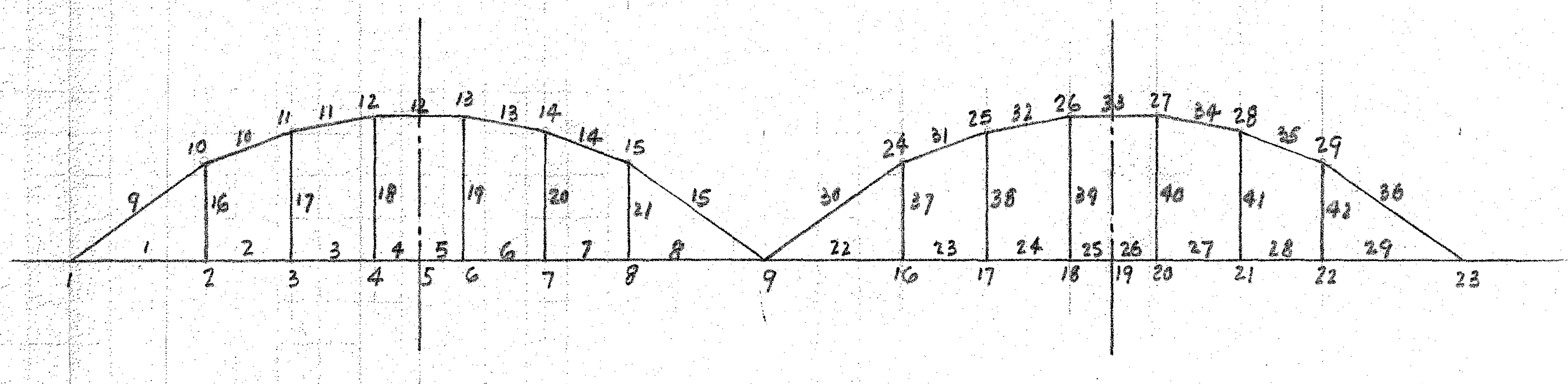


BY T.T.

JOB PUTAH CREEK BR. (23C-92)

DATE 8/17/78

SUBJECT FLOOR BEAM



BY T.T. JOB PUTAH CREEK BRIDGE (23C-92) SHEET ___ OF ___

DATE 7/21/78 SUBJECT ARCH RATING

DS-D15 (REV. 3-75) DIST., CO., RTE., P.M.

SECTION PROPERTIES:

ARCH SECTION:

WIDTH OF ARCH = 2'-3"
DEPTH " " = 3'-0"
} CONSTANT DIM.

$A = 2.25 \times 3 = 6.75 \text{ FT}^2$ $I = 2.25 \times 3^3/12 = 5.06 \text{ FT}^4$

$DL = 6.75 \times .15 = 1.013^{K}/'$

HANGER SECTION:

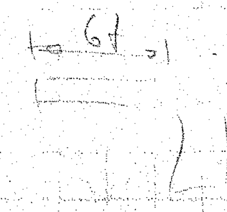
WIDTH OF HANGER = 1'-3"
DEPTH OF HANGER = 1'-8"

$A = 1.25 \times 1.67 = 2.084 \text{ FT}^2$ $I = 1.25 \times 1.67^3/12 = 0.483 \text{ FT}^4$

DL HANGERS 16 & 21 = $2.084 \times .15 \times 9.6' \pm = 3.0^{K}$
" " 17 & 20 = " $\times 14.4' \pm = 4.5^{K}$
" " 18 & 19 = " $\times 16.5' \pm = 5.2^{K}$

EXTERIOR GIRDER SECTION:

WIDTH OF SECTION = 1'-11"
DEPTH OF SECTION = 5'-3"



$A = 1.92 \times 5.25 = 10.08 \text{ FT}^2$ (13.430 FT^2)
$I = 1.92 \times 5.25^3/12 = 23.15 \text{ FT}^4$ (36.071 FT^4)
$DL = 10.08 \times .15 = 1.512^{K}/'$

BY T. T. JOB PUTAH CREEK BR. (23C-92) SHEET ___ OF ___

DATE 7/21/78 SUBJECT ARCH RATING

DS-D18 (REV. 3-75) DIST. CO. RTE. P.M.

SUPERIMPOSED DEAD LOADS:

SLAB + RAILING:

ASSUME 9" SLAB & RAILING IS UNIFORM DL ON EXTERIOR GIRD.

DL SLAB = 0.75 × 10.0 × 0.15 = 1.125 K/'
DL RAILING = ASSUME DL = .250
1.375 K/'

FLOOR BEAMS:

ASSUME 2'-0" STEM OF FLOOR BEAM ARE CONCENTRATED MEMBER LOADS ON THE EXTERIOR GIRDER.

DL FLOOR BEAM = 2.0 × 10.0 × .15 = 3.0 K

CROSS GIRDERS (STRUTS):

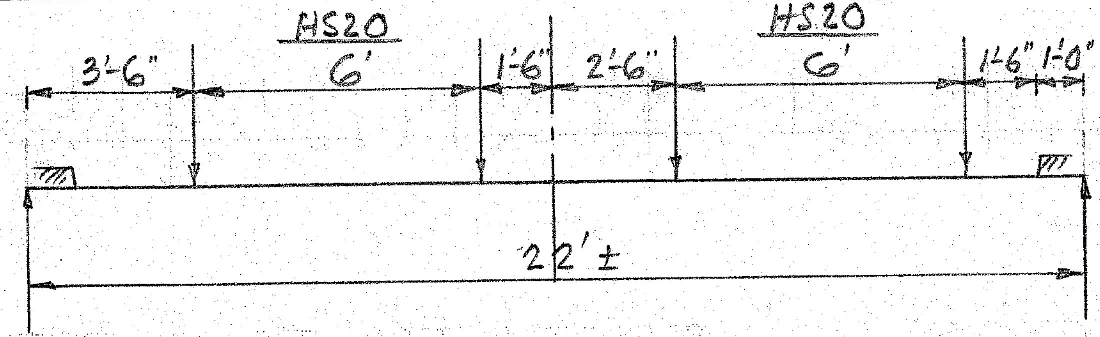
ASSUME CROSS GIRDER REACTIONS ARE JT. LOADS AT JTS. 11 & 14.

ASSUME WIDTH OF STRUT = 1'-8"
" AVERAGE DEPTH OF STRUT = 2.25'

DL REACTION = 1.67 × 2.25 × 10.0 × .15 = 5.64 K

LL + IMPACT:

HS20 LOADING:



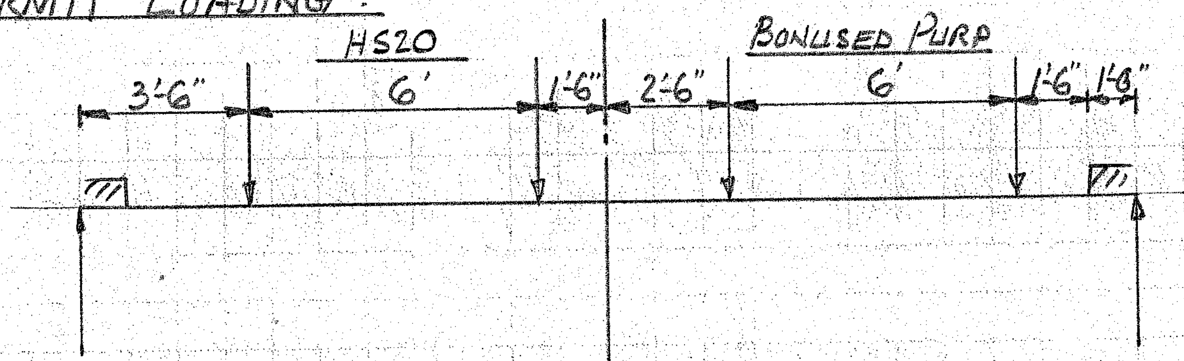
RT REACTION:

RT LANE = (13.5 + 19.5)/22 = 1.50 WHEEL LINES
LT LANE = (3.5 + 9.5)/22 = 0.59 " "
2.09 " "

ASSUME IMP. = 50/99.5+125 = 22.3%

NO. OF LANES OF HS20 = $\overset{1.045}{2.09} \times 1.223/2$ = 1.278 LANES

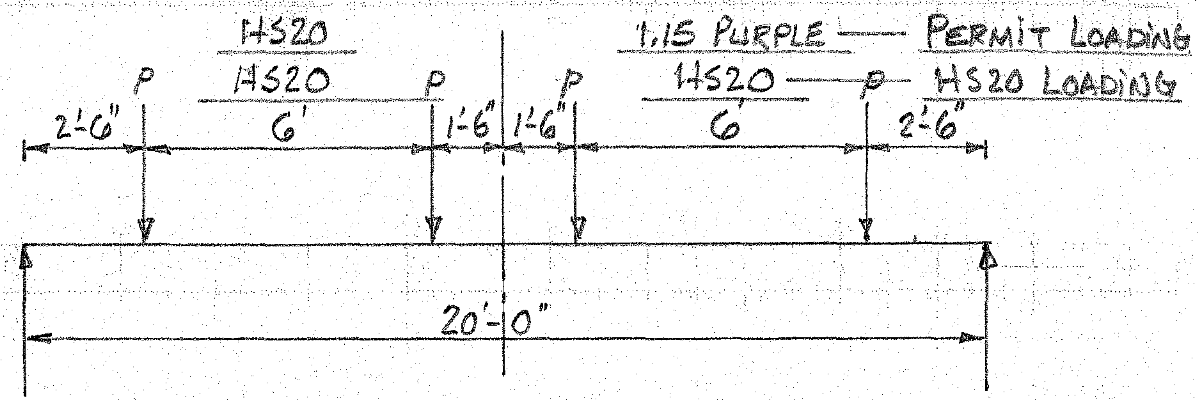
PERMIT LOADING:



RT REACT. BONUSED PURP = 1.15(13.5 + 19.5)/22 = 1.725 WHEEL LINES

LANES OF PERMIT VEH. = $\overset{.863}{1.725} \times 1.223/2$ = 1.055 LANES

COMBINE WITH FRACTION OF HS20 = .59/2.09 = .282 × HS20

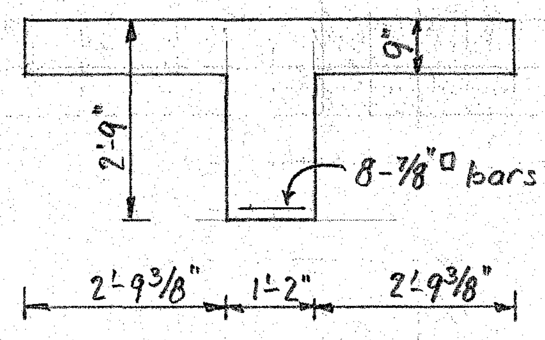


$f_y = 33$ KSI $\quad f'_c = 2.5$ KSI

$b = 6'\text{-}8\tfrac{3}{4}" = 80.75"$ $\quad A_s = 6.13 \text{ in}^2$

ASSUME $d = 30"$ & $\phi = 0.90$

a = depth of COMPRESSIVE SECTION



ASSUMED SECTION OF FLOOR BEAM

RESISTING MOMENT OF SECTION:

$$a = A_s f_y / 0.85 f'_c b = 6.13 \times 33 / .85 \times 2.5 \times 80.75 = 1.18 \text{ in.}$$

$$M_R = \phi A_s f_y (d - a/2) = 0.90 \times 6.13 \times 33 (30 - 1.18/2) 1/12 = 446.2^{\,\prime K}$$

DEAD LOAD:

$$DL \text{ OF FLANGE} = 0.75 \times 2.78 \times 0.15 \times 2 = 0.63^{K}/'$$
$$DL \text{ OF GIRDER} = 1.17 \times 2.75 \times 0.15 = 0.48$$
$$\Sigma = 1.11^{K}/'$$

$$DL \text{ MOMENT} = 1.11 \times 20^2/8 = 55.5^{\,\prime K}$$

BY T.T. JOB PUTAH CREEK BR. (23C-92) SHEET ___ OF ___

DATE 7/21/78 SUBJECT FLOOR BEAM DIST., CO., RTE., P.M.

DS-D18 (REV. 3-75)

LIVE LOAD (HS20):

ASSUME WHEEL LINE REACT. ON FLOOR BM. = 32.0^K
THEN $P = 16.0^K$

MOM. @ ℄ BEAM = $32.0 \times 10 - 16(1.5 + 7.5) = 176^{'K}$

ASSUME IMP. = 30%

LL + IMP MOM = $176 \times 1.3 = 228.8^{'K}$

LIVE LOAD (PERMIT VEHICLE):

ASSUME $P = 24.0^K$

RT. REACT. OF BONUSED PURP. = $1.15 \times 24(11.5 \times 17.5)/20 = 40.02^K$

" " " HS20 = $16(2.5 + 8.5)/20 = 8.8^K$

℄ MOM OF BONUSED PURPLE = $[40.02 \times 10 - 1.15 \times 24(1.5 + 7.5)]1.3 = 197.3^{'K}$

℄ MOM. OF HS20 = $8.8 \times 10 = 88^{'K}$

RATING FACTORS:

R.F. (INV) = $446.2 - 1.3 \times 55.5 / \tfrac{5}{3} \times 1.3 \times 228.8 = 0.75$

R.F. (OPER) = $446.2 - 1.3 \times 55.5 / 1.3 \times 228.8 = 1.26$

R.F. (PURP) = $446.2 - 1.3 \times (55.5 + 88.0) / 1.3 \times 197.3 = 1.01$

BY T. T. JOB PUTAH CREEK BR. (23C-92) SHEET ____ OF ____

DATE 9/7/78 SUBJECT EXTERIOR GIRDER

DS-D18 (REV. 3-75) DIST.. CO.. RTE.. P.M.

SECTION CAPACITY:

$A_s = 15.24\ in^2$ (12-#10 BARS) $d = 60''$ $b = 23''$

$f'_c = 2.5$ ksi $f_y = 33$ ksi

$a = 15.24 \times 33 / .85 \times 2.5 \times 23 = 10.29''$

$M_u = .9 \times 33 \times 15.24\,(60 - 10.29/2) \times 1/12 = 2069^{1K}$

CHECK MEMBER ④ - JT. 5:

DL MOM = 301.9^{1K} HS20 Mom = 296.3^{1K}

PURPLE MOM. = 342.2^{1K}

R.F. (INV.) = $2069 - 1.3 \times 301.9 / 1.3 \times 5/3 \times 296.3 = 2.61$

R.F. (OPER) = $2069 - 1.3 \times 301.9 / 1.3 \times 296.3 = 4.35$

R.F. (PERMIT) = $2069 - 1.3 \times 301.9 / 1.3 \times 342.2 = 3.77$

CHECK MEMBER ④ - JT. 4:

DL MOM. = 240.9^{1K} HS20 Mom. = 296.3^{1K} P13 Mom = 372.7^{1K}

R.F. (INV.) = $2069 - 1.3 \times 240.9 / 1.3 \times 5/3 \times 296.3 = 2.74$

R.F. (OPER) = $2069 - 1.3 \times 240.9 / 1.3 \times 296.3 = 4.56$

R.F. (PERMIT) = $2069 - 1.3 \times 240.9 / 1.3 \times 372.7 = 3.62$

CHECK MEMBER ③ - JT. 3:

DL MOM. = 133.9^{1K} HS20 Mom. = 375.3^{1K} P13 Mom. = 492.0^{1K}

R.F. (INV.) = $2069 - 1.3 \times 133.9 / 1.3 \times 5/3 \times 375.3 = 2.33$

R.F. (OPER) = $2069 - 1.3 \times 133.9 / 1.3 \times 375.3 = 3.88$

R.F. (PERMIT) = $2069 - 1.3 \times 133.9 / 1.3 \times 492 = 2.96$

IST	ROUTE	COUNTY	STRU. NO	POSTMILE	RATING WIDTH-FT	STRU TYPE	YR ORIG CONST	
10	CORD	23 C	0092	.	024.0	CG T BEAM	23	JUL. 20, 1978

	RATING FACTOR	PT	SPAN	ULT MOM CAP TOP IN COM	ULT MOM CAP BOT IN COM	POS HS20 MOMENT	NEG HS20 MOMENT	POS PURP MOMENT	NEG PURP MOMENT	DEAD LOAD MOMENT	SECONDARY MOMENT
NV	2.50	5	1	6646.1	0.0	956.8	0.0	1123.2	0.0	1134.6	0.0
PER	4.16	5	1	6646.1	0.0	956.8	0.0	1123.2	0.0	1134.6	0.0
URP	3.54	5	1	6646.1	0.0	956.8	0.0	1123.2	0.0	1134.6	0.0

HE NUMBER OF AXLES ON THE TRUCK THAT CAUSES THE PURPLE RATING FACTOR IS 13.
'C = 1.00 FY = 33.

F THE REPORTED ULTIMATE MOMENT CAPACITY IS 0, IT WAS DETERMINED NOT TO BE CRITICAL

BRIDGE ACROSS PUTAH CREEK
RATING OF T-BEAM APPROACH SPAN
NO A.C. 7/78

IST	ROUTE	COUNTY	STRU. NO	POSTMILE	RATING WIDTH-FT	STRU TYPE	YR ORIG CONST	
10	CORD	23 C	0092	.	024.0	CG T BEAM	23	JUL. 20, 1978

INFLUENCE LINE FOR CRITICAL INVENTORY RATING POINT SPAN 1 10TH POINT 5

MEM NO	LEFT	.1	.2	.3	.4	.5	.6	.7	.8	.9	RIGHT
1	0.0	1.800	3.600	5.400	7.200	9.000	7.200	5.400	3.600	1.800	0.0

THE CRITICAL OPERATING RATING POINT IS THE SAME AS THE CRITICAL INVENTORY RATING POINT

THE CRITICAL PURPLE RATING POINT IS THE SAME AS THE CRITICAL INVENTORY RATING POINT

BRIDGE ACROSS PUTAH CREEK
RATING OF T-BEAM APPROACH SPAN
NO A.C. 7/78

RATING SUMMARY OF ARCH BRIDGES

DIST.	RTE	COUNTY	STRU No.	P.M.	WIDTH	ARCH CLASSIFICATION	YR. ORIG. CONST.
10	Co Rd	Sol	23C-92		11' (HALF WIDTH)	OPEN SPANDREL (TIED ARCH)	1923

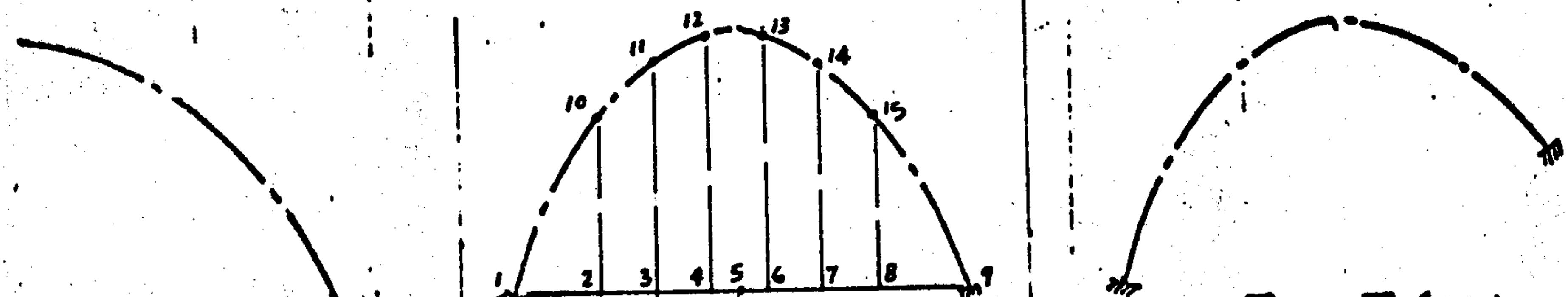
RATED BY: T. Tsukiji DATE: 7/78

RATING	FACTOR	JT.	SP. TYPE	SPAN No.	AXIAL DL	DL MOM.	AXIAL LL	LL MOM	AXIAL LOAD CAP.
INV.	3.01	1	II	1	350.6^K	-114.9^IK	128.2^K	-179.2^IK	958.0^K
OPER.	5.02	1	II	1	350.6^K	-114.9^IK	76.9^K	-107.5^IK	958.0^K
PURPLE	3.10	1	II	1	350.6^K	-114.9^IK	234.8^K	-58.9^IK	1403.5^K

F'_c = 2.5 ksi F_y = 33 ksi ϕ = 0.70

NAME OF STRUCTURE: Putah Creek Br. (23C-92)

COMMENTS: Span 1 rated; Span 2 assumed similar



TYPE I SPAN   TYPE II SPAN   TYPE III SPAN

BRIDGE NO. 23C-92

LOCATION 10-SOL-Co Rd.

NAME PUTAH CREEK BR.

ARCH RIB
RATING

	FACTOR	VEHICLE	CRIT. LOC.	
INVENTORY	0.75	HS 15.0	FLOORBEAM	HS 20.
OPERATING	1.26	HS 25.2		HS 34
PERMIT				
5 AXLE		PPPPP		P
7 AXLE				
9 AXLE				
11 AXLE				
13 AXLE	1.01			P
LEGAL				
TYPE 3				
TYPE 3S2				
TYPE 3-3				
AC=				

SAFE LOAD CAP 6

RATED BY: T. TSUKIJI

DATE: 7/78

V 2/7

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
BRIDGE REPORT
DS-M58 (REV. 1/75)

Bridge No. 23C-92
Other No.
P.U.C. No.
Location 10-Sol-FAS 1112
Dist - Co - Rte - PM - City

REVISED ORIGINAL REPORT

Date of Investigation April 25, 1975

Name PUTAH CREEK (on Stevenson Bridge Rd.)

Lat. 38°-32.4' Long. 121°-51.1'

STRUCTURAL DATA AND HISTORY

Year Built 1923 By Solano & Yolo Counties Contract No. Unknown

Date of Revisions

Designed by: B.D. ☐ Counties Plans Avail. @ BD

Description: Reinforced concrete tied arches with RC (5) girder approach spans on 2-column piers and RC piles, seat abutments on spread footings.

Spans 1@40', 2@108', 1@40'

Length 298' Skew none Design LL county-medium

Ratings: Inventory Operating Permit Legal Loads Only

DESCRIPTION – ON STRUCTURE

Bridge Width 20'

Total Width 24.2' Lanes 2 Tracks none

Median none Rail Type conc. (1000)

Vert. Clearance over deck 14'-3" Appr. Rdwy. Width 18.5'

Wearing Surface none Deck Seal none

Alignment very sharp horiz. curve to bridge then tangent

DESCRIPTION – UNDER STRUCTURE

Roadway Section – –

Clearances: Vert. – – Horiz.; – – Lt. – – Rt.

Lanes – – Tracks – – Pumpplant: None ☐ See Br. No. – –

Facilities Crossed Creek

cc: FCH/dn

(1) CON'T.

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
BRIDGE REPORT
DS-M58A (REV. 1/75)

Bridge No. 23C-92

Date April 25, 1975

DESCRIPTION — HYDRAULICS

Channel sand and gravel

Navigable: Yes ☐ No ☒ Clearances: Vert. .... Horiz. ....

MAINTENANCE

Custodian County Sol & Yolo   Owner County Sol & Yolo

ORIGINAL CONDITION RATING		ORIGINAL APPRAISAL	
Deck	4-F4	Overall	4-F4
Superstructure	6-F6	Deck Geometry	4
Substructure & Pipes	5-F5	Underclearances Vert.	5
Channel & Channel Protection	5-F5	Horiz.	3
Retaining Walls	5-F5	Safe Load Capacity	5-F5
Approach Rdwy. Alignment	3-F3	Waterway Adequacy	6
Estimated Remaining Life	20	Approach Rdwy. Alignment	3

Widenable? Yes ☐ No ☒ Conditional ☐   Action Required by ~~District~~ County: Yes ☒ No ☐

Refer to bridge reports dated 7/9/71 and 11/19/73.

CONDITION OF STRUCTURE

Some new spalls due to over width trucks on this structure were noted at the following locations; First Truss--the left posts #1, #3 and #4; Second Truss--the right and left #4 posts; the right railing at the northerly end of the Second Truss.

All spalls were 6" diameter or less.

Three feet of the timber piles under Bent #3 are exposed, one foot of which is above water and subject to alternating wet and dry cycles.

This structure remains in a generally fair condition as of the date of this investigation.

RECOMMENDATION

The piles at Bent #3 should be protected from exposure.

Frank C. Heggli
Frank C. Heggli

REGISTERED PROFESSIONAL ENGINEER
FRANK C. HEGGLI
No. 11248
CIVIL
STATE OF CALIFORNIA

State of California

SUPPLEMENTARY BRIDGE REPORT

Bridge No. 23C-92

Location 10-Sol-FAS 1112-27.0
Dist - Co - Rte - PM - City

Date of Investigation November 19, 1973

Name PUTAH CREEK (Stevenson Bridge Road)

CONDITION RATING: APPRAISAL RATING:

Deck 4 Superstructure 6 Substr. & Pipes 5 Overall 4

Channel & Channel Protection --- Retaining Walls ---

Widenable? Yes ☐ No ☒ Conditional ☐

Action Required by District: Yes ☐ No ☒

CONDITION OF STRUCTURE

The structure remains in a generally fair condition as of the date of this investigation.

Frank C. Heggli
FRANK C. HEGGLI
C 11248

FCH/jf

OH-OS M19 (REV 8/73) (1)

25C92

GENERAL COLUMN ANALYSIS

Page 1 of 2

Name G.W. Heller

Phone 5-5408

IDENT (DIST / GROUP / BATCH)	PROBLEM	SOURCE (DIST. / UNIT)	CHARGE (DIST. / UNIT)	EXP. AUTH. (GEN. LED / SUB-ACCT / MAINT W/O / ROUTE NO. / WORK ORDER NO.)	SPECIAL DESIGNATION (USE WHEN APPLICABLE) (PREFIX / BRIDGE NUMBERS / PARCEL OR CONTR. NO / DMG. RPTS. LABR. CDS. ENCR. PMTS. / SUB-JOB NOS. / LOCATIONS)	OBJECT	PROGRAM NUMBER
14500	0.01	14602	14601	96170	1423C092	661	BDEØ13

S/C 1230, 1191, 1192 S/C 1230

REINFORCING BAR DATA

Bar Size	Coordinates of a Single Bar or One End of a Row of Bars: X (0.01 inch)	Y (0.01 inch)	Number of Bars in Row	Coordinates of the Other End of the Row: X (0.01 inch)	Y (0.01 inch)
1.0	260	260	6	2440	260
1.0	260	3340	6	2440	3340

S/C 1191

INSTRUCTIONS

REINFORCING BAR DATA:

The bar size is to be indicated by the numerical designation of the size of bar.

The coordinates of each individual bar may be given, or if you so desire, only the coordinates of the end bars of a row may be specified. The number of bars in a row must be indicated if the coordinates of the end bars are given.

The number of reinforcement bars must not exceed 500.

For a pile footing problem, the letter "P" is to be placed in the column reserved for Bar Size. The pile locations are to be indicated by the coordinates.

Insert the letter "S" into Bar Size to indicate a spread footing problem. Leave the rest of the line blank.

CONCRETE DATA:

The first and last concrete point must be the same.

All coordinates are to be positive.

A maximum of 300 points may be used to describe the section.

The coordinates are to be given in a clockwise sequence unless a void is involved, in which case the coordinates of the void are to be given in a counter-clockwise sequence.

Enter the coordinates horizontally from left to right on a line, using as many lines as required to indicate all the points.

For a pile footing problem, the concrete data is to be left blank.

CONCRETE DATA

Coordinates of Concrete Points (0.01 inch)

X	Y	X	Y	X	Y	X	Y	X	Y
000	000	000	3600	2700	3600	2700	000	000	000

S/C 1192 FORM H-BD D 26 (REV. 8/68)

23C-92

GENERAL COLUMN ANALYSIS

Page 2 of 2

Name G.W. Heller

Phone 5-5408

IDENT			PROBLEM
DIST	GROUP	BATCH	
14	5	00	001

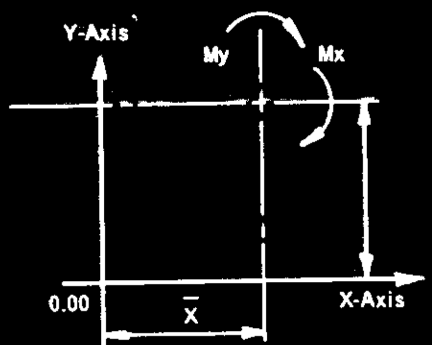
S/C 1193

LOAD DATA

Load Number	MOMENTS (kip-ft.) My About Y-Axis	MOMENTS (kip-ft.) Mx About X-Axis	AXIAL FORCE P(kips)	DISTANCE TO MOMENT CENTER (0.01 inch) $\overline{X}$	DISTANCE TO MOMENT CENTER (0.01 inch) $\overline{Y}$	Compressive N' Value	Tension N Value	Stresses*	COMMENTS
1	1	348	528	1350	1800				DL LLL LLP13
2	1	319	500	↓	↓				DL LLL LLP 7
3	1	261	425	↓	↓				DL LLL LLL

S/C 1193

FORM H-BD D 43 (REV. 8/68)



INSTRUCTIONS

LOAD DATA:

Mx and My represents the moments about the X and Y axes respectively. These moments are to be applied so as to act in the direction shown. A minimum moment of 1 kip-ft. must be applied on a column or footing.

$\overline{x}$ and $\overline{y}$ are the coordinates of the point about which the moments are assumed to act. For a rectangular column or footing, this point would, in general, be the centroid of the member.

N' is the ratio of the Moduli of Elasticity of the compression steel to the concrete. A value of 10 will be used if the columns are left blank.

N is the ratio of the Moduli of Elasticity of the tension steel to the concrete. A value of 10 will be used if the columns are left blank.

For footing analyses, leave the N' and N columns blank.

Check the column noted "Stresses*" if the stresses at all the concrete points in compression and at all the reinforcement bars are desired. This option should be used with discretion as the time for calculation is increased.

Stevenson Br. 23C-92

	LOADING DATA				RESULTS (UNITS POUNDS & INCHES)			
	MOMENTS (KIP FT)	AXIAL FORCE (KIPS)	MOMENT CENTER COORDINATES (INCHES)	MODULI RATIO	MAXIMUM CONCRETE STRESS	ALLOWABLE CONCRETE STRESS	MAXIMUM STEEL STRESSES COMPRESSION	TENSION
LOAD NO. 1 COMMENTS *DL LLL LLP13 *								
MY	1.	528.	X 13.50	N' 10	1029.	> 962	9486.	15.
MX	348.		Y 18.00	N 10	X 27.00		X 24.40	X 2.60
					Y 0.0		Y 2.60	Y 33.40
LOAD NO. 2 COMMENTS *DL LLL LLP 7 *								
MY	1.	500.	X 13.50	N' 10	956.	956	8831.	
MX	319.		Y 18.00	N 10	X 27.00		X 24.40	
					Y 0.0		Y 2.60	
LOAD NO. 3 $\boxed{\text{COMMENTS *DL LLL LLL *}}$								
MY	1.	425.	X 13.50	N' 10	796.	950	7365.	
MX	261.		Y 18.00	N 10	X 27.00		X 24.40	
					Y 0.0		Y 2.60	

Member 1 - Joint 1   DL + 2 LL+I of 325 legal loads

f_c = 796 psi

f_s = 7365 psi - compression.

BY GWH JOB Stevenson Br. 23C-92 SHEET 0 OF
DATE 2-3-72 SUBJECT Bridge Operating Rating 10-Sd-FAS 1112
DIST., CO., RTE., P.M.

Summary

This bridge can carry all combinations of Legal Loads.

H-BD

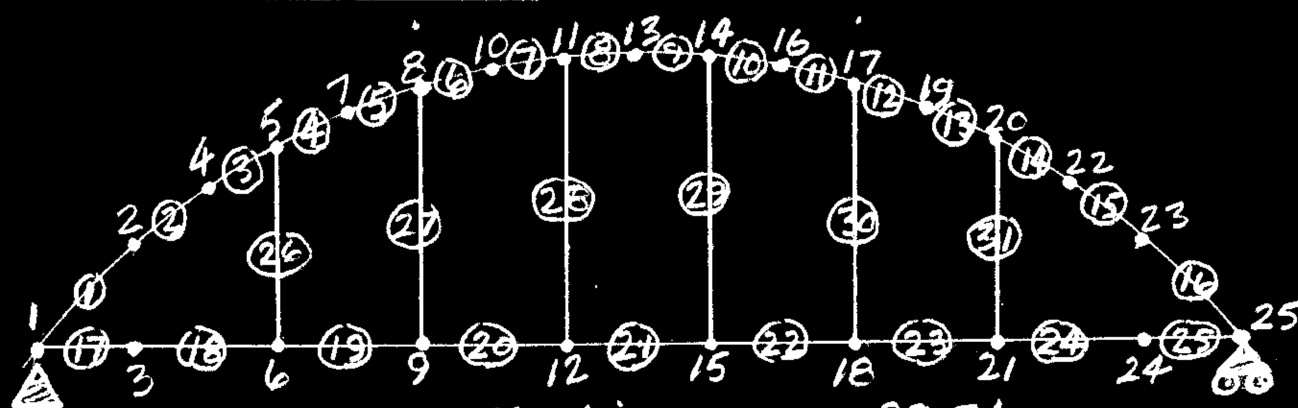
GWH Stevenson Br. 23C-92 SHEET 1 OF

DATE 12-21-71 SUBJECT Bridge Operating Rating 10-Sol-FAS 1112

DIST. CO. RTE. P.M.

For coordinate layout of structural members for STRUDL II input see SHEET NO 1 - "RUMSEY BRIDGE (Stevenson Bridge - actual local name - plans may be general but are field checked for site) - REINFORCED CONCRETE BRIDGE ACROSS PUTAH CREEK FOR YOLO & SOLANO COUNTIES"

Tied Arch Analysis



effective span = 99.5'

$I = 0.222$; Vertical members no M

Section Properties

Members 1 - 16 = Arch Section

$$I = \frac{bd^3}{12} = \frac{2.25 \times \overline{3.00}^3}{12} = 5.06 \text{ ft}^4$$

$$A = 2.25 \times 3.00 = 6.75 \text{ ft}^2$$

$$W = 6.75 \times 0.150 = 1.01 \text{ k/ft}$$

Members 17 - 25 = Tied Lower Chord Section

$$I = \frac{bd^3}{12} = \frac{1.92 \times \overline{5.25}^3}{12} = 23.15 \text{ ft}^4$$

$$A = 1.92 \times 5.25 = 10.08 \text{ ft}^2$$

$$W = 10.08 \times 0.150 = 1.51 \text{ k/ft}$$

BY GLUH JOB Stevenson Br. 23C-92 SHEET 2 OF

DATE 12-21-71 SUBJECT Bridge Operating Rating 10-Sol-FAS-1112

DIST. CO. RTE. P.M.

Members (26) - (31) = Hanger Section

$I = \frac{bd^3}{12} = \frac{1.25 \times 1.67^3}{12} = 0.49 \ ft^4$

$A = 1.25 \times 1.67 = 2.09 \ ft^2$

$W = 2.09 \times 0.150 = 0.31 \ K/ft$

Contributory Dead Load

Girder & Floor System

$Deck(\frac{1}{2}) = 0.75 \times 10.17 \times 0.15 = 1.14 \ K/ft.$

$\underset{(1/2)}{Floor\ Beam} = \frac{1.17 \times 10.17 \times 2.00 \times 0.15}{6.70} = 0.53 \ K/ft$

$\underset{(1\ side)}{Bridge\ Rail} = \frac{2.75 \times .67 \times .50 \times 4 \times .15}{5} = 0.11 \ K/ft.$

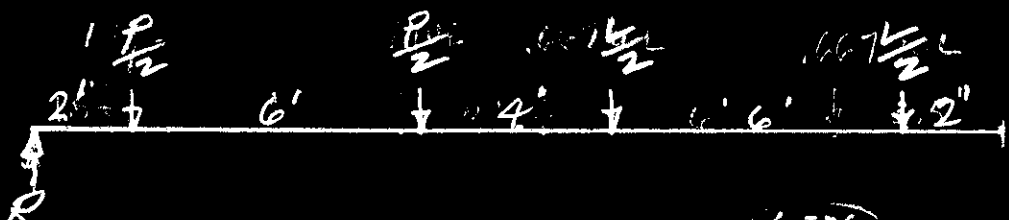
Lateral Tie Strut @ Pt. (8) & (17)

$Ave\ \frac{1}{2}\ Strut = 1.67 \times 2.25 \times 10.0 \times 0.15 = 5.64 \ K @ Pt.$

Live Load Systems

Standard legal load 3-S-2 1 lane load in STRUDL program,

Purple P-7 26k 18' 48k 18' 48k 18' 48k 1 lane in STRUDL program + 6 axles for P-13



$R_p = \frac{18 \times \frac{P}{2} + 12 \times \frac{P}{2}}{20} = \frac{10.75 P}{20}$ load

$R_L = \frac{8 \times \frac{L}{2} + 2 \times \frac{L}{2}}{20} = 0.25 L$ load

H-BD-D18

BY GWH JOB Stevenson Br. 23C-92 SHEET 3 OF

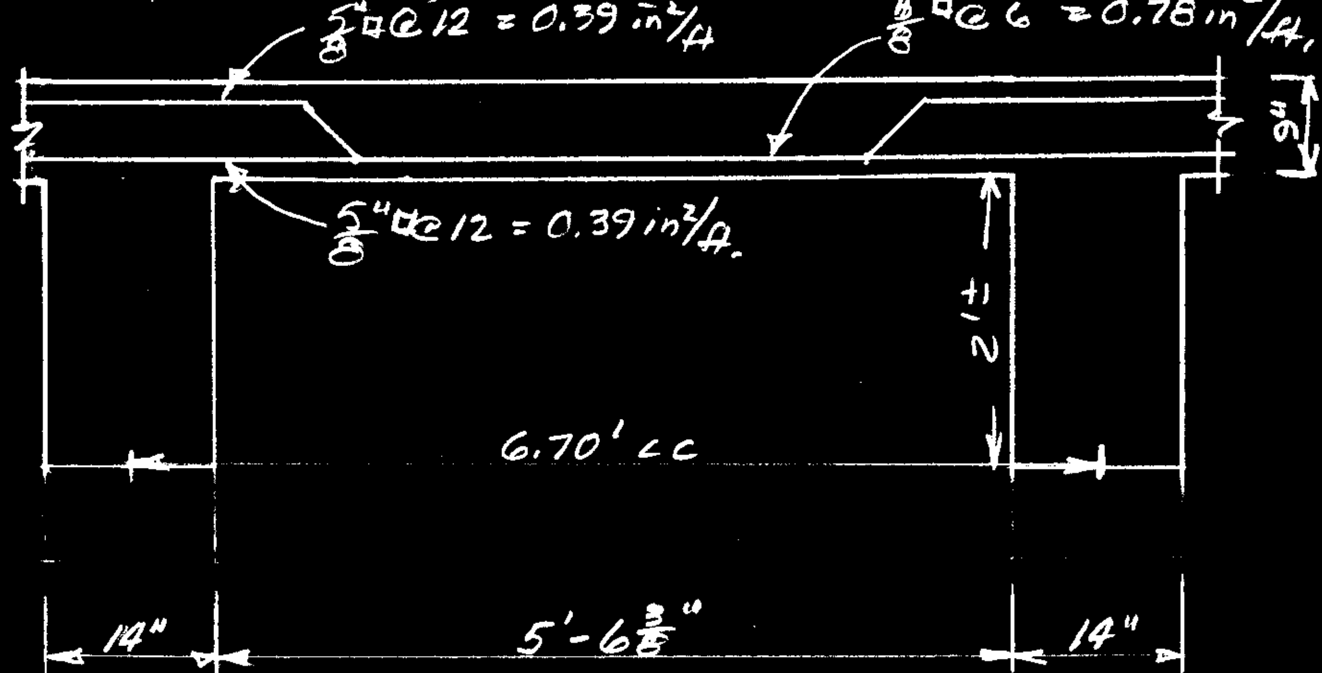
DATE 1-13-72 SUBJECT Bridge Operating Rating 10-Sol-FAS 1112

DIST. CO. RTE. P.M.

Floor System Analysis

Deck Slab - arch span

Longitudinally Reinforced



Distribution wheel loads, $E = 4 + 0.06S$

$E = 4 + 0.06 \times 5.53 = 4.33$ ft.

Purple truck M for 5.53' span = 38.5'k lane, SBM 28k axle

Since floor slab continuous over transverse floor beams assume M equals results from 5 span influence lines as conservative est. for range of stress. $I = 30\%$ lane over 2E

$DLM + = .0405 \times .75 \times .150 \times \overline{6.70}^2 = 0.20$ 'k/ft

$DLM - = .0846 \times .75 \times .150 \times \overline{6.70}^2 = 0.43$ 'k/ft

$LL+I + \; (28^k \text{ axle}) = .1704 \times 28.0 \times \frac{6.70 \times 1.30}{8.66} = 4.80$ 'k/ft

(.166)

$LL+I - \; (2\text{-}24^k) = (.082 + .084) \times 24 \times \frac{6.70 \times 1.30}{8.66} = 4.01$ 'k/ft

H-BD-D18

BY GLWH JOB Stevenson Br. 23C-92 SHEET 4 OF

DATE 1-14-72 SUBJECT Bridge Operating Rating 10-Sel-FAS 1112 DIST., CO., RTE., P.M.

For analysis purposes with Olivetti 101 computer for Double Reinforced beam program

① − Over trans beam: $n = 10$ in $d = 7$ in $A'_s = 0.39\ in^2$
$b = 12$ in $A_s = 0.39\ in^2$
$d' = 2$ in
$M = 4.44^k = 53,280$ in lbs

$f_c = \underline{772}$ psi

$f'_s = \underline{1077}$ psi tension

$f_s = \underline{21203}$ psi tension

olivetti underwood programma 101 olivetti

	V
10	S n
0 • 39	S A'_s
0 • 39	S A_s
7	S d
2	S d'
12	S b
1 • 8696	A ◊ kd
12	S b
53280	S M
772 • 6828	A ◊ f_c
10	S n
− 1077 • 8540	A ◊ f'_s
21203 • 3172	A ◊ f_s

② + Midspan: $n = 10$ $d = 7$ $A_s = 0.78\ in^2$
$b = 12$ in
$M = 5.00^k = 60,000$ in lbs

$f_c = \underline{663}$ psi

$f_s = \underline{12,431}$ psi tension

101 olivetti underwood programma 101 oli

	V
10	S n
0 • 00	S A'_s
0 • 78	S A_s
7	S d
2	S d'
12	S b
2 • 4358	A ◊ kd
12	S b
60000	S M
663 • 4164	A ◊ f_c
10	S n
2373 • 8960	A ◊ f'_s
12431 • 0916	A ◊ f_s

∴ Deck slab OK

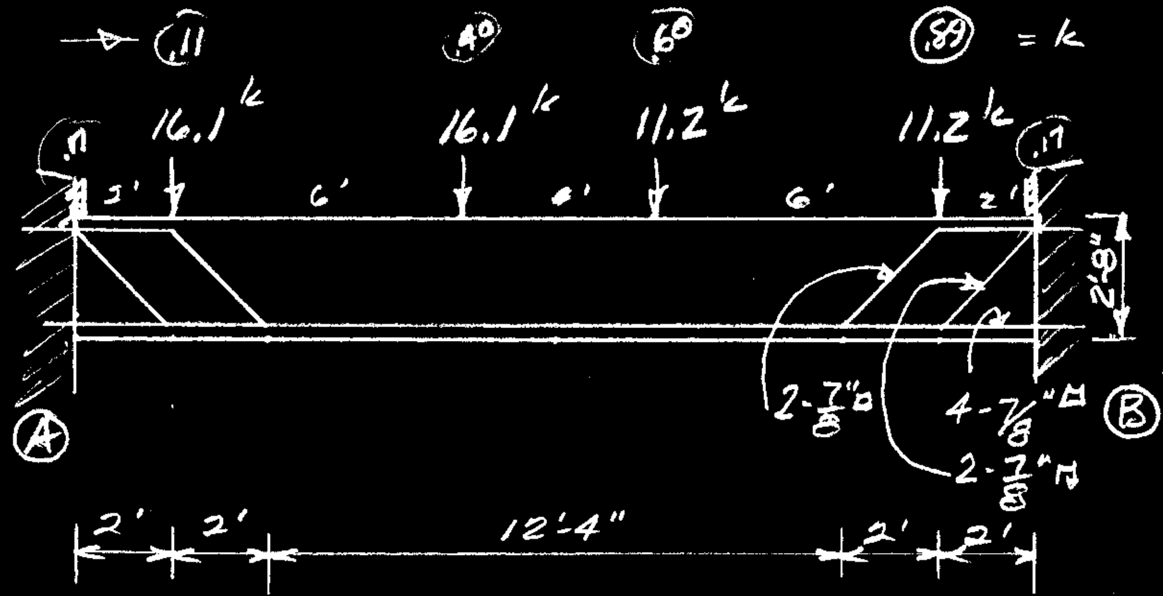
for all loads P,G,O,L,

△ OSP H-BD-D18

By GWH Job Stevenson Br. 23C-92 Sheet 5 of

Date 1-14-72 Subject Bridge Operating Rating 10-Sol-FAS 1112

Transverse Floor Beam - arch span



$L = 20.33'$

FEM @ (A) $PL = 16.1 \times 20.33$ (327.31) @ (B) $PL = 11.2 \times 20.33$ (227.70)

	@ (A)		@ (B)	
.11	.0871 × 327.31 =	28.51	.0108 × 327.31 =	3.53
.40	.1440 × 327.31 =	47.13	.0960 × 327.31 =	31.42
.60	.0960 × 227.70 =	21.86	.1440 × 227.70 =	32.79
.89	.0108 × 227.70 =	2.46	.0871 × 227.70 =	19.83
		99.96'k		87.57'k
	+I =	129.95'k	+I =	113.84'k

use conc. load coef.

SBM

16.1 @ .11	= .0979 × 327.31 =	32.04'k	+I	= 41.65'k
16.1 @ .40	= .2400 × 327.31 =	78.55'k	✓	= 102.12'k
11.2 @ .60	= .2400 × 227.70 =	54.65'k	✓	= 71.04'k
11.2 @ .89	= .0979 × 227.70 =	22.29'k	✓	= 29.00'k

SBV @ (A) @ (B)

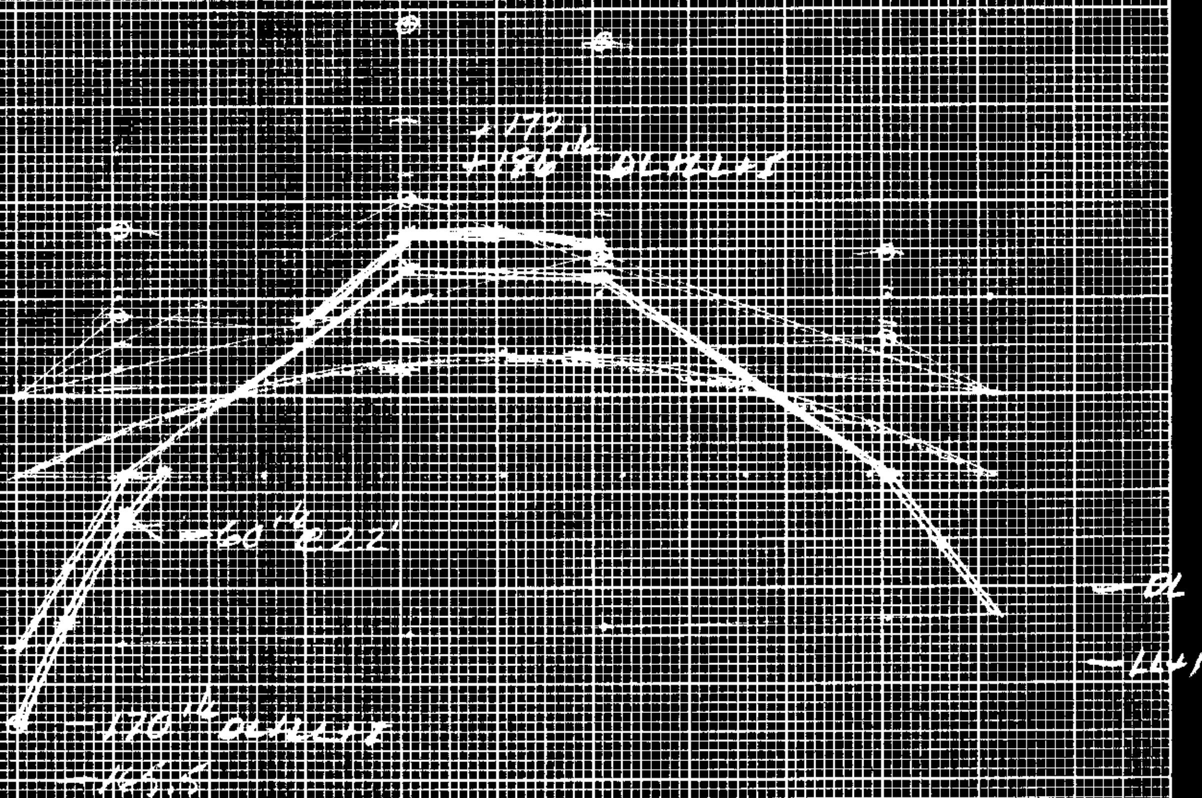
.89 × 16.1 =	14.33	.11 × 16.1 =	1.77
.60 × 16.1 =	9.66	.40 × 16.1 =	6.44
.40 × 11.2 =	4.48	.60 × 11.2 =	6.72
.11 × 11.2 =	1.23	.89 × 11.2 =	9.97
	29.70^k		24.90^k

DLM: $FEM = \frac{wl^2}{12} = \frac{1.22 \times 20.33^2}{12} = \cancel{42}$ 37.5'k (1.09) $V = 1.09 \times 10.16 = 11.1^k$

$SBM = \frac{wl^2}{8} = \cancel{\frac{1.22 \times 20.33^2}{8}} = \cancel{63}$ 56'k

H-8D-D18

Stevenson Br. 23C-92 10-241-EAS-1112
1-17-72

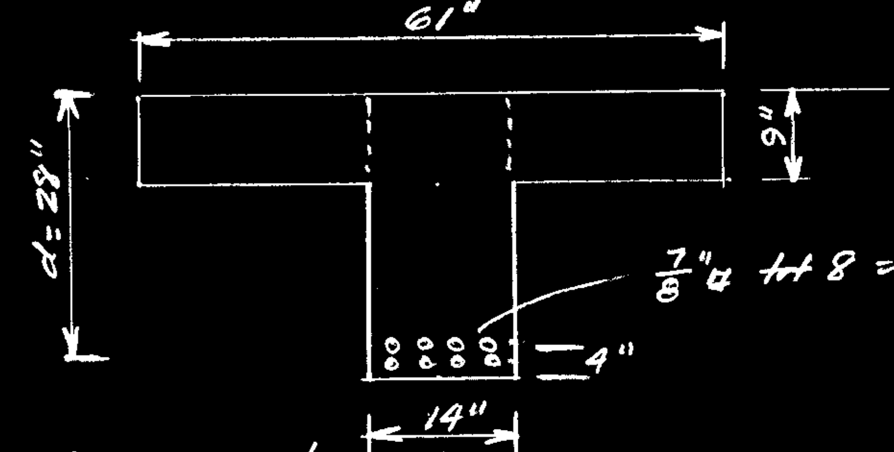


Transverse Floorbeam in Arch span.

horiz.: 1" = 4'
vert.: 1" = 100k

X-Section - (see sheet 5 for longitudinal view)

Effective flange width $= \frac{20.33}{4} = 5.08' = 61"$



+M near center span.
From plans: $b = 61"$ effective; $b' = 14$ in
$d = 28$ in.; $t = 9$ in.
$A_s = 6.12\ in^2$; $n = 10$ assume
$M = +186$ ft. kips. 2,232,000

ACI-RC Design Hdbk

$$m = \frac{nA_s}{b'd} + \frac{(b-b')t}{b'd} = \frac{10 \times 6.12}{14 \times 28} + \frac{(61-14)9}{14 \times 28}$$

$$= 0.156 + 1.079 = 1.235$$

$$q = \frac{nA_s}{b'd} + \frac{(b-b')t}{b'd} \times \frac{\frac{1}{2}t}{d} = 0.156 + 1.079 \times \frac{.5 \times 9}{28}$$

$$= 0.156 + 0.173 = .329$$

$$k = \sqrt{m^2 + 2q} - m$$

$$= \sqrt{2.183} - 1.235$$

$$= 1.477 - 1.235 = \underline{.242}$$

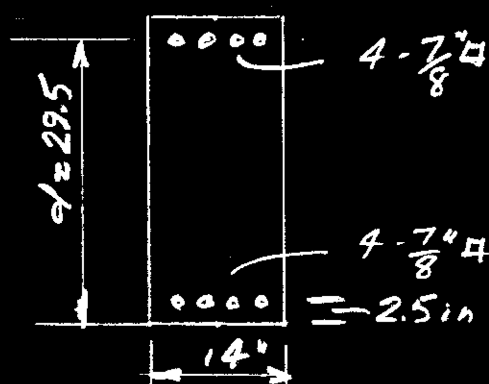
Since $A's = 0$ $z = \frac{1}{3}$; $j = 1 - \frac{1}{3}k = 1 - \frac{.242}{3} = .919$

$f_s = \dfrac{12000 M}{j d A_s} = \dfrac{12000 \times 186}{.919 \times 28 \times 6.12} = 14{,}173 \text{ psi}$ ok

$f_c = \dfrac{f_s}{n} \times \dfrac{k}{1-k} = \dfrac{14173}{10} \times \dfrac{.242}{.758} = 452 \text{ psi}$ ok

$-M$ @ Support (note d & d' for $-M$ configuration)

$n = 10$
$b = 14$ in
$d' = 2.5$ in
$d = 29.5$ in
$A'_s = 3.06 \text{ in}^2$
$A_s = 3.06 \text{ in}^2$



$-M = 170^{k} = 2{,}040{,}000$ in. lbs. @ face
$-M = 60^{k}$ 720,000 in lbs @ 2' out.

programma 101 olivetti underwood programr.

@ 2' out		@ face	
10	S n	10	S n
3.06	S A'_s	3.06	S A'_s
1.53	S A_s	3.06	S A_s
29.5	S d	29.5	S d
2.5	S d'	2.5	S d'
14	S b	14	S b
5.3731	A◇ kd	7.4418	A◇ kd
14	S b	14	S b
720000"#	S M	2,040,000"#	S M
382.6506	A◇ fc	832.6719	A◇ fc
10	S n	10	S n
4092.2120	A◇ f's	11058.8760	A◇ f's
17192.2131	A◇ fs	24691.1046	A◇ fs

ok if beam not cracked clear thru.

½ # stir @ 12" V_s

$v = \dfrac{V}{jbd} = \dfrac{}{.875 \times 14 \times 29.5} =$ [illegible] $\left.\begin{matrix}2^{k}\\9^{k}\end{matrix}\right\} 81^{k} > 40.8^{k}$

∴ Transverse beam is OK for all loads PG, OL.

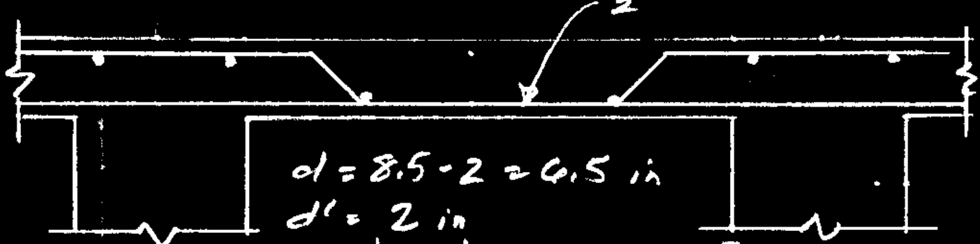
Deck Slab - approach spans

Transverse reinforcement

Purple load as single axle = 28^k or 14^k WL
From AASHO Case A main reinf. ⊥ to traffic

$$LLM = \frac{(S+2)}{32} P_{purple} = \frac{(4.25+2)}{32} 14 = 2.73^{'k}/\text{ft width}$$

Continuous over 3 = .8 × 2.73 = ± LLM = $2.19^{'k}$



℄ $A_s = 0.50 in^2$
@ Support $A_s = 0.25 in^2$; $A'_s = .25 in^2$

$$DLM_{\pm} = \frac{wS^2}{10} = \frac{0.11 \times 4.25^2}{10} = 0.20^{'k}/ft$$

$$DL+LL+I = 0.20 + 2.19 + 0.66 = 3.05^{'k} = 36600^{in\#}$$

olivetti underwood programma 101

		@ ℄ S		over girder	
		10	S n	10	S n
		0·00	S A'_s	0·25	S A'_s
		0·50	S A_s	0·25	S A_s
		6·5	S d	6·5	S d
		2	S d'	2	S d'
		12	S b	12	S b
		1·9477	A◊ kd	1·5537	A◊ kd
		12	S b	12	S b
f_c		36600	S M	36600	S M
< 1200 psi		535·2953	A◊ f_c	737·4411	A◊ f_c
		10	S n	10	S n
f_s		-287·4760	A◊	-4236·5940	A◊ f'_s = tension
< 25000 psi		12511·2954	A◊ f_s	23476·8957	A◊ f_s

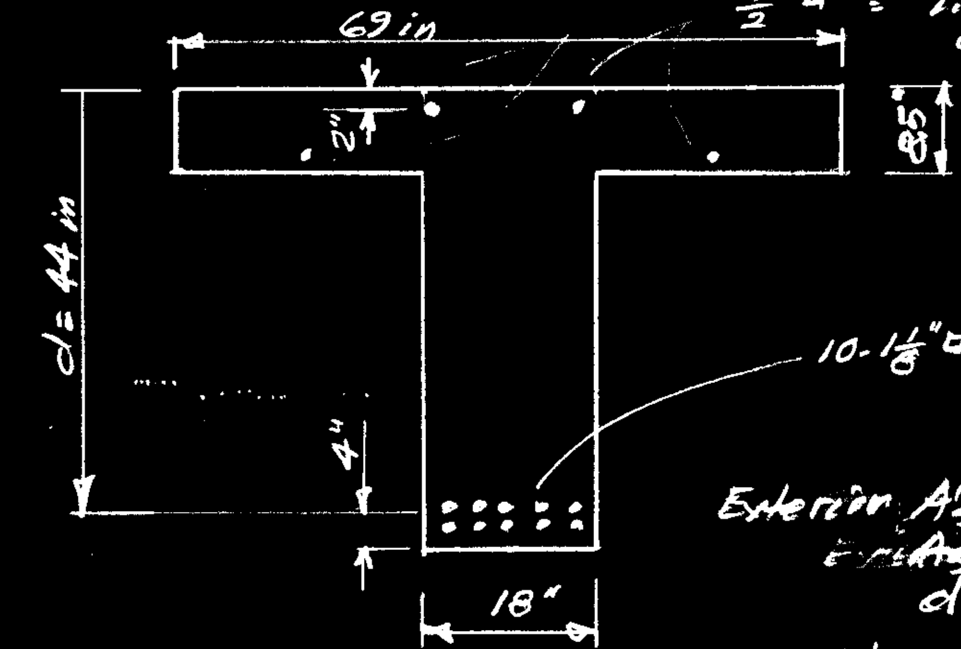
∴ Slab OK for all loads

"T" Beam Girders - approach spans - effect span = 37.5'

X-Section $DL = 1.35^{k}/ft$

Interior -

Effective flange = 69 in



Exterior $A'_s = 0.50 in^2$

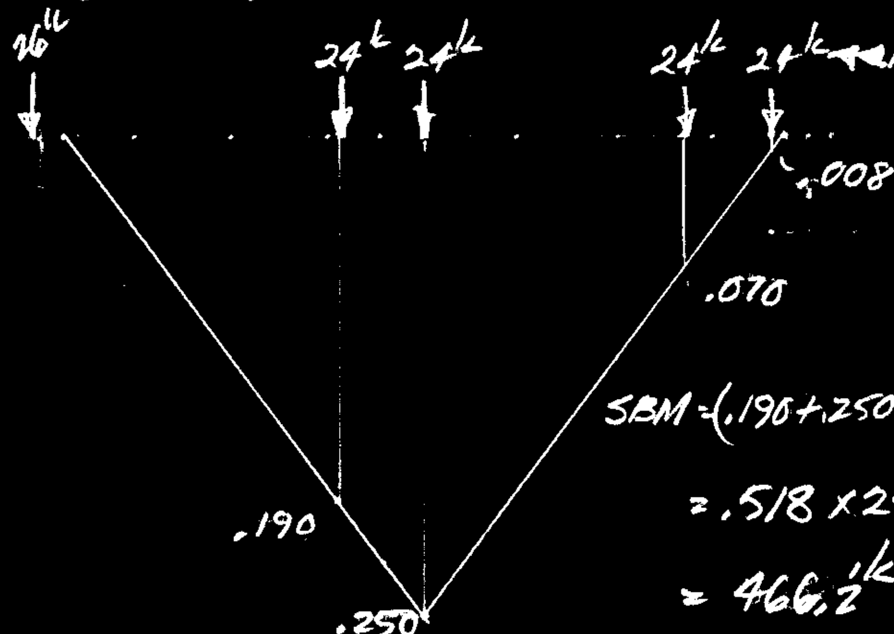
~~Ext.~~ $A_s = 8\text{-}1\frac{1}{8} = 10.13 in^2$

$d = 59 in$

$$M_{DL} = \frac{wl^2}{8} = \frac{1.35 \times 37.5^2}{8} = 237.3^{k'}$$

$$M_{LL/girder} = \frac{S}{6}\,\frac{\text{Purple Lane}}{2} = \frac{5.75}{6} \times \frac{466.2}{2} = 223.4^{k'}$$

$$I = 0.30 \times 223.4 = 67.0$$



SBM = (.190 + .250 + .070 + .008) PL

= .518 × 24 × 37.5

= 466.2'k Purple lane.

By GWH Job Stevenson Br. 23C-92 Sheet 11 of

Date 1-25-72 Subject Bridge Operating Rating 10-Sel-FAS 1112 (Dist., Co., Rte., P.M.)

$DLM + LLM + I = 237.3^k + 223.4^k + 67.0^k = 527.7'^k/\text{girder}$

From plans: $d = 44"$; $d' = 4.25"$; $b = 69"$; $b' = 18"$

$t = 8.5"$; $n = 10$

$A'_s = 1.00\ in^2$; $A_s = 12.65\ in^2$

olivetti underwood prograr

$f_c = \underline{414}$ psi

$f'_s = \underline{5089}$ psi

$f_s = \underline{12352}$ psi

44	S d
8.5	S t
69	S b
1.00	S A's
12.65	S As
18	S b'
2.5400	A0 $\bar{X}$
11.0400	A0 kd

11.04	S kd	
69	S b	
18	S b'	
8.5	S t	
1.00	S A's	
44	S d	
12.65	S As	
527.7	S M	
413.7394	A0 fc	>1300
4.25	S d'	
5089.2940	A0 f's	>25,000
12382.2210	A0 fs	>25,000

$V = \frac{(36.7 + 32.[illegible]75 + [illegible].25)24}{37.5} = 65.2^k/\text{lane}$

$LLV/\text{girder} = \frac{5.75}{6} \times \frac{65.2}{2} = 31.2^k$

$DLV/\text{girder} = 18.75 \times 1.35 = 25.3^k$

$I = 0.30 \times 31.2 = 9.4^k$

$V_{DL+LL+I} = 65.9^k$

$v = \frac{V}{bjd} = \frac{65.9}{18 \times .875 \times 44} = 0.095\ ^k/in^2$

$v_c = 0.150 \times 18 \times 0.875 \times 44 = 104.^k$

Stirrups $\frac{3}{8}"\phi$ @ 18

$V_s = \frac{2 \times .11 \times 25 \times .875 \times 44}{18} = 15.0^k$

$V_c + V_s = 119^k > 65.9^k$ OK if diagonal cracking not excessive

∴ Interior beams OK for all loads.

△ OSP

H-BD-D18

BY Guv lt JOB Stevenson Br. 23C-92 SHEET 12 OF

DATE 1-25-72 SUBJECT Bridge Operating Rating 10-Sel-FAS-1112

DIST., CO., RTE., P.M.

Exterior girder - $DL = 1.4^{k}/ft.$

$$\text{Fraction of WL} = \frac{2.4}{5.75} = 0.417$$

$$M_{LL}/\text{girder} = 0.417 \times \frac{466.2}{2} = 97.2'^{k}$$

$$M_{DL} = \frac{wl^2}{8} = \frac{1.40 \times 37.5^2}{8} = 246.1'^{k}$$

$$M_{DL+LL+I} = 246.1 + 97.2 + 29.2 = 372.5'^{k} = 4{,}470{,}000''^{\#}$$

Section of girder as double reinf. beam

$b = 18''$; $d = 63 - 4 = 59\text{ in}$; $d' = 2\text{ in}$; $n = 10$

$A'_s = 0.50\text{ in}^2$; $A_s = 10.13\text{ in}^2$; $b = 18$

101 olivetti underwood programma 101

10	S	n
0·50	S	A'_s
10·13	S	A_s
59	S	d
2	S	d'
18	S	18
20·3789	A◊	kd
18	S	b
4470000	S	M
444·1703	A◊	f_c
10	S	n
8011·5820	A◊	f'_s
8417·7004	A◊	f_s

$f_c = 444$ psi

$f'_s = 8012$ psi

$f_s = 8418$ psi

∴ Exterior girder OK

Main Tied Arch Spans

Maximum Load Condition

DL from STRUDL II program is per arch ring unit.

LL lanes from STRUDL II are given as full lane load. From sheet 2 - lane loads are per arch ring = 0.75 for adjacent lane & 0.25 for far lane
With $I = \frac{50}{L+125} = \frac{50}{99.5+125} = 0.223$

Then $LL\ factor_{adjacent} = 1.222 \times 0.75 = \underline{0.9165}$

& $LL\ factor_{far} = 1.222 \times 0.25 = \underline{0.3055}$

Arch Section

From STRUDL output Max Load Condition occurs @ Member 1, Joint 1

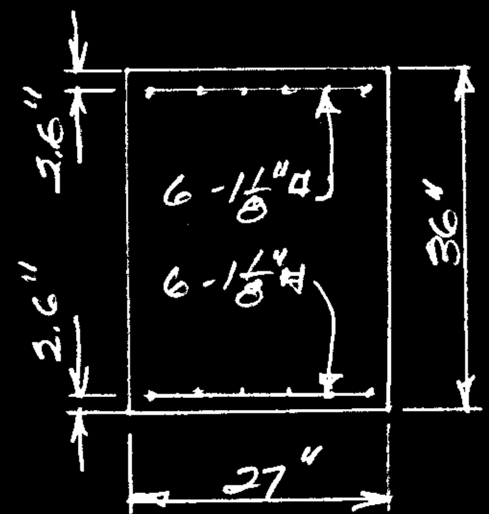
		Axial (compression)	M
1	DL	342.35 × 1 = 342.35	203.59 × 1 = 203.59
	LL_{352}	67.45 × .3055 = 20.61	46.73 × .3055 = 14.28
	LL_{P13}	180.01 × .9165 = 165.00	142.05 × .9165 = 130.19
		$\underline{528^k}$	$\underline{348'^k}$
2	LL_{P7}	150 × .9165 = 137.48	108.04 × .9165 = 99.02
		$\underline{500^k}$	$\underline{319'^k}$
3	LL_{352}	67.45 × 1.222 = 82.42	46.73 × 1.222 = 57.10
		$\underline{425^k}$	$\underline{261'^k}$

By GWH Job Stevenson Br. 23C-92 Sheet 14 of

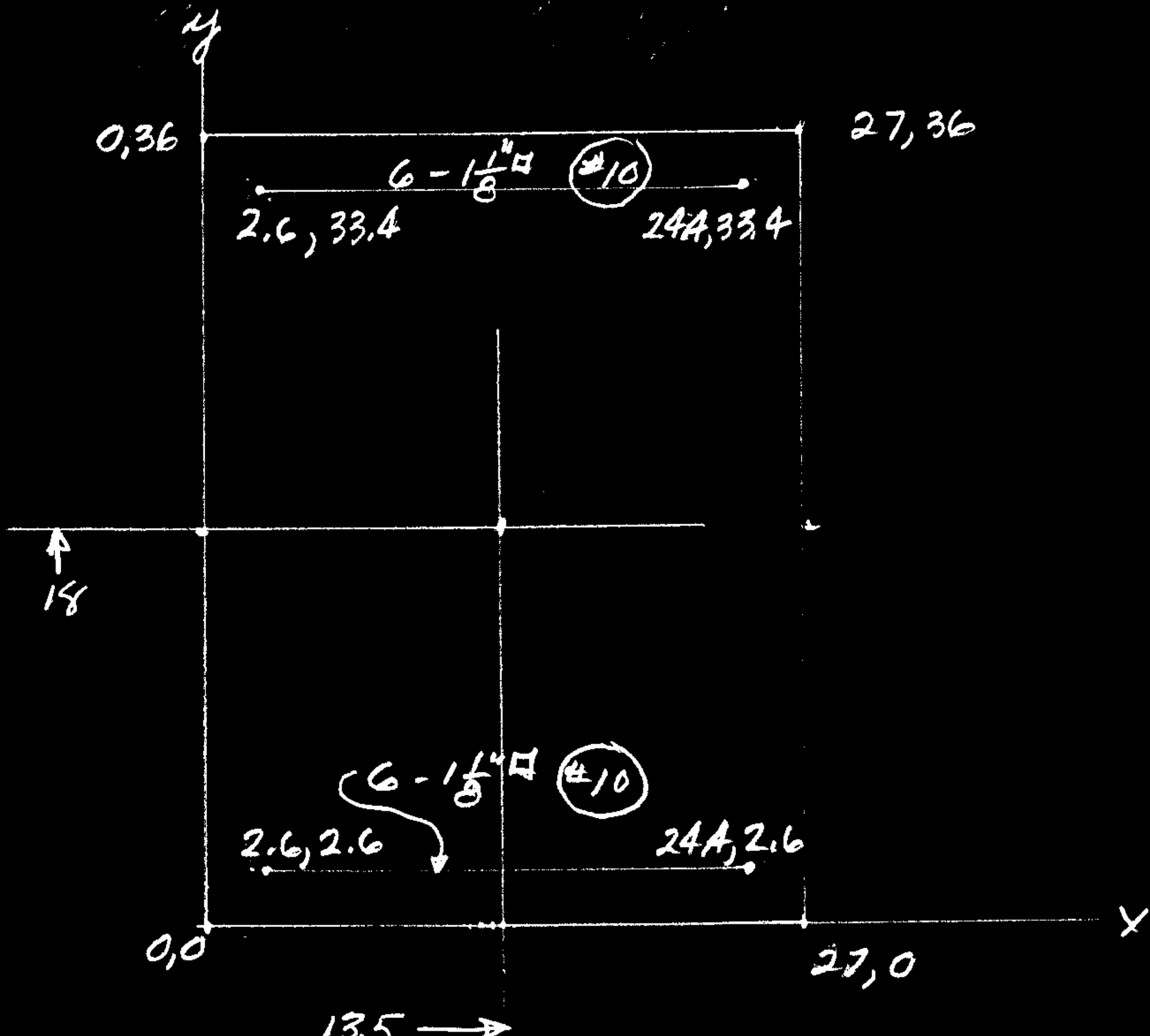
Date 1-31-72 Subject Bridge Operating Rating 10-Sol-FAS 1112

Dist. Co. Rte. P.M.

X-Section - Arch Ring - from plans.

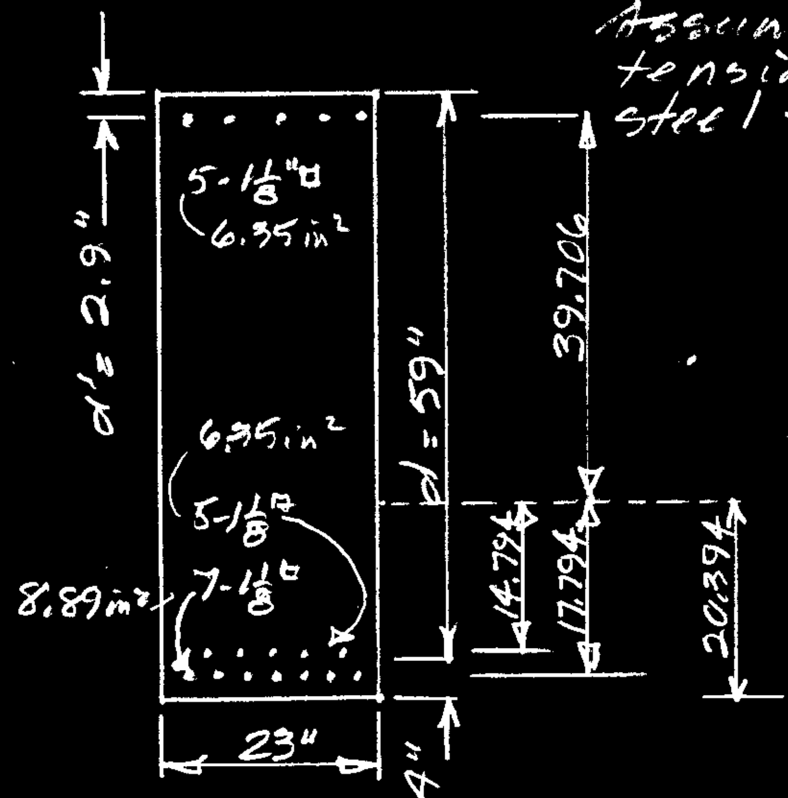


Assume 2" clear
n = 10



Coordinates for General Column Analysis

Tied Chord Section 23"x63"



Tension & moment on section
Assume concrete takes no tension. Then c.g. of reinf. steel l = $\frac{(60.1\times5)+(5.6\times5)+(2.6\times7)}{(5+5+7)}$

$= \frac{346.7}{17} = 20.394$

$I_s = 6.35\times\overline{39.706}^2$
$+6.35\times\overline{14.794}^2$
$+8.89\times\overline{17.794}^2$

= 10011.2
1389.8
2814.8

$I_s = 14215.8$ in⁴

From STRUDL output Max Load Condition occurs @ Member 20, Joint 9 for DL+LL_{352} + LL P13 or P7 and @ Member 20, Joint 12

		Axial (tension)		M
@J.9	DL	276.8 × 1	= 276.8	160.8 × 1 = 160.8
	LL_{352}	56.4 × .3055	= 17.2	213.8 × .3055 = 65.3
	LL P13	149.5 × .9165	= 137.0	353.9 × .9165 = 324.3
			431.0	550.4
	LL P7	124.2 × .9165	= 113.8	332.0 × .9165 = 304.3
			407.8	530.4
@J.12	DL	276.8 × 1	= 276.8	250 × 1 = 250
	LL_{352}	56.4 × 1.222	= 68.9	160.3 × 1.222 = 195.9
			345.7	445.9

For $DL + LL_{352} + LL_{P13} + I$ & $N = \underline{431^k}$ tension; $M = \underline{550.4'^k}$

$$f_{s\,top\,M} = \frac{Mc}{I} = \frac{550.4 \times 12000 \times 39.71}{14215.8} = 18450 \text{ psi}$$

$$f_{s\,bot\,M} = \text{''} = \frac{550.4 \times 12000 \times 17.79}{14215.8} = 8265 \text{ psi}$$

$$f_{s\,direct} = \frac{N}{A_{s\,total}} = \frac{431 \times 1000}{21.59} = 19{,}963 \text{ psi}$$

Tension top steel = 19,963 − 18450 = 1513 psi

Tension bot steel = 19,963 + 8265 = 28228 psi

Too high

For $DL + LL_{352} + LL_{P13} + I$ & $N = \underset{tension}{\underline{407.8^k}}$; $M = \underline{530.4'^k}$

$$f_{s\,top\,M} = \frac{Mc}{I} = \frac{530.4 \times 12000 \times 39.71}{14215.8} = 17{,}779 \text{ psi}$$

$$f_{s\,bot\,M} = \text{''} = \frac{530.4 \times 12000 \times 17.79}{14215.8} = 7965 \text{ psi}$$

$$f_{s\,direct} = \frac{N}{A_{s\,total}} = \frac{407.8 \times 1000}{21.59} = 18{,}888 \text{ psi}$$

Tension top steel = 18888 − 17779 = 1109 psi

Tension bot steel = 18888 + 7965 = 26853 psi

Too high

For $DL + LL_{3S2} + I$: $N = \underset{\text{tension}}{345.7^k}$; $M = 445.9$

$$f_{s\,top\,M} = \frac{Mc}{I} = \frac{445.9 \times 12000 \times 39.71}{14215.8} = 14947 \text{ psi}$$

$$f_{s\,bot\,M} = \text{"} = \frac{445.9 \times 12000 \times 17.79}{14215.8} = 6696 \text{ psi}$$

$$f_{s\,direct} = \frac{N}{A_{s\,total}} = \frac{345.7 \times 1000}{21.59} = 16012 \text{ psi}$$

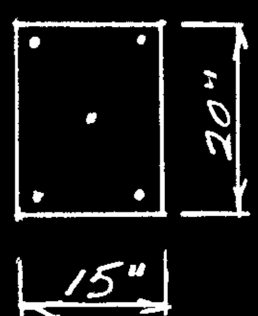
Tension top steel = 16012 - 14947 = 1065 psi

Tension bot steel = 16012 + 6696 = 22708 psi

OK for Legal Loads

v-shear no problem - ample bent $1\frac{1}{8}$" bars provided @ each joint of lower members.

Arch Hanger Section 15"x20"



$5 - 1\frac{1}{8}$"□ = 6.35 in² = A_s

In STRUDL analysis the hangers were considered free ended — take no moment.

∴ All stress is tension and taken by the steel only

From STRUDL output Max Load Condition occurs @ Member 26 - Joint 5

Axial (tension)

①	DL	70.57 × 1.0	=	70.57^k
	+			
	LL_{352}	18.52 × .3055	=	5.66
	+			
	LL_{P13}	48.58 × .9165	=	44.52
				$\underline{\underline{120.75}}^k$
②	$DL + LL_{352}$			
	LL_{P7}	39.64 × .9165	=	36.33
				$\underline{\underline{112.56}}^k$
③	$DL + LL_{352}$			
	LL_{352}	18.52 × 1.222	=	22.63
				$\underline{\underline{93.20}}^k$

① $f_{s_{\text{tension direct}}} = \dfrac{P}{A_{s_{\text{total}}}} = \dfrac{120.75}{6.35} = 19.02^k/in^2$ ✓

② $=$ ✓ $= \dfrac{112.56}{6.35} = 17.73^k/in^2$ ✓

③ $=$ ✓ $= \dfrac{93.20}{6.35} = 14.68^k/in^2$ ✓

∴ Hangers OK for all loads. ✓

General Note: Temperature was considered in the Arch Spans - results; stresses negligible for a CTE .00021 / unit.

STATE OF CALIFORNIA – BUSINESS AND TRANSPORTATION AGENCY – DEPARTMENT OF PUBLIC WORKS – DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b	b	ADDRESS GROUP	BATCH
	$	145	03

SUBSYSTEM NAME | SOURCE DIST. UNIT | CHARGE DIST. UNIT | EXPENDITURE AUTHORIZATION | SPECIAL DESIGNATION WHEN APPLICABLE | SEQUENCE ØØØ1

Statement (cols 1–63)	Sequence
LOADING 30 'LEGAL 3 S 2'	710
MEM 19 LOAD FORCE Y CON P -8.0 L 9.0	711
20 -16.0 8.0	712
20 -16.0 12.0	713
22 -16.0 10.0	714
23 -16.0 2.0	715
LOADING 16 'LEGAL 3 S 2'	716
MEM 23 LOAD FORCE Y CON P -8.0 L 3.0	717
22 -16.0 4.0	718
22 -16.0 0.0	719
20 -16.0 2.0	720
19 -16.0 10.0	721
LOADING 45 'PURPLE 13'	722
MEM 17 LOAD FORCE Y CON P -24.0 L 3.25	723
17 -24.0 7.75	724
19 -24.0 1.50	725
19 -24.0 6.00	726
20 -24.0 7.50	727
20 -24.0 12.00	728
22 -24.0 1.50	729
22 -24.0 6.00	730
23 -24.0 7.50	731
23 -24.0 12.00	732

BY:
CHECK:
DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:
NAME Bill Heller
PHONE 5-5408 DATE 1-27-72

VERIFY V-15

PAGE 1 OF 1

S/C 2470

HCS-329 (REV. 1/1/70)

b	$	b	ADDRESS DIST.	ADDRESS GROUP	BATCH
			1	45	03

ICES - (Con't)

```
     MEM 25 LŌAD FŌRCE Y CŌN P -24.0 L  1.5       733
      ↓  25  ↓     ↓   ↓  ↓  ↓ -24.0 ↓  6.0       734
LŌADING  22 'PURPLE 13'                           735
     MEM 25 LŌAD FŌRCE Y CŌN P -24.0 L  4.5       736
      ↓  25  ↓     ↓   ↓  ↓  ↓   ↓   ↓  0.0       737
      ↓  23  ↓     ↓   ↓  ↓  ↓   ↓   ↓ 10.5       738
      ↓  23  ↓     ↓   ↓  ↓  ↓   ↓   ↓  6.0       739
      ↓  22  ↓     ↓   ↓  ↓  ↓   ↓   ↓  4.5       740
      ↓  22  ↓     ↓   ↓  ↓  ↓   ↓   ↓  0.0       741
      ↓  20  ↓     ↓   ↓  ↓  ↓   ↓   ↓ 10.5       742
      ↓  20  ↓     ↓   ↓  ↓  ↓   ↓   ↓  6.0       743
      ↓  19  ↓     ↓   ↓  ↓  ↓   ↓   ↓  4.5       744
      ↓  19  ↓     ↓   ↓  ↓  ↓   ↓   ↓  0.0       745
      ↓  17  ↓     ↓   ↓  ↓  ↓   ↓   ↓  6.25      746
      ↓  17  ↓     ↓   ↓  ↓  ↓   ↓   ↓  1.75      747
LŌAD LIST ALL                                     800
LIST FŌRCES LŌADS REACTIŌNS DISPLACEMENTS         810
```

S/C 2470

VERIFY V-15

STATE OF CALIFORNIA – BUSINESS AND TRANSPORTATION AGENCY – DEPARTMENT OF PUBLIC WORKS – DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b	$	ADDRESS DIST.	GROUP	BATCH
		14	S	03

64 65 66 67 68 69 70 71 72

SUBSYSTEM NAME	b	b	SOURCE DIST.	SOURCE UNIT	CHARGE DIST.	CHARGE UNIT	EXPENDITURE AUTHORIZATION	SPECIAL DESIGNATION WHEN APPLICABLE	b	SEQUENCE
		$	14	602	14	60	1961701	423C092		ØØØ1

1-5	6-10	11-15	16-20	21-25	26-30	31-35	36-40	41-45	46-50	51-55	56-60	61-63	73-76
MOVE	LOAD	BOTH	TRUCK	NP 7	26 1	5.75	24 4.	5 24	13.50	24 4	.5 –		730
	24 13	.50 24	4.5	24 1	3.50	24 4.	5 24	13.50	24 4	.5 24	–		740
	13.50	24 4	.5 24										741

BY:

CHECK:

DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:

NAME George W. Heller

PHONE 5-5408 DATE 1-24-72

VERIFY

PAGE 1 OF 1

S/C 2470

HCS-329 (REV. 1/1/70)

STATE OF CALIFORNIA – BUSINESS AND TRANSPORTATION AGENCY – DEPARTMENT OF PUBLIC WORKS – DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b	$	b	ADDRESS DIST.	GROUP	BATCH
			14	5	03

SUBSYSTEM NAME	b	$	b	SOURCE DIST.	SOURCE UNIT	CHARGE DIST.	CHARGE UNIT	EXPENDITURE AUTHORIZATION	SPECIAL DESIGNATION WHEN APPLICABLE	b	SEQUENCE
		$		14	602	14	601	961701	423C092		0001

	SEQUENCE
STRUDL 'STEVEN' 'BRIDGE 23C 92 10 SOL FAS 1/1/2'	10
TYPE PLANE FRAME	20
UNITS KIP FEET	30
JOINT COORDINATES	40
1 X 0.00 Y 0.00 SUPPORT	50
2 7.75 6.60	60
3 7.75 0.00	70
4 13.75 10.90	80
5 19.75 14.00	90
6 19.75 0.00	100
7 25.75 16.60	110
8 31.75 18.45	120
9 31.75 0.00	130
10 37.75 19.80	140
11 43.75 20.60	150
12 43.75 0.00	160
13 49.75 20.90	170
14 55.75 20.60	180
15 55.75 0.00	190
16 61.75 19.80	200
17 67.75 18.45	210
18 67.75 0.00	220
19 73.75 16.60	230

BY:

CHECK:

DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:

NAME George W. Heller

PHONE 5-5408 DATE 1-11-72

VERIFY

S/C 2470

PAGE 1 OF 4

HCS-329 (REV. 1/1/70)

ICES - (Con't)

SEQUENCE CONTINUED

```
    20    79.75    14.00                                                        240
    21    79.75     0.00                                                        250
    22    85.75    10.90                                                        260
    23    91.75     6.60                                                        270
    24    91.75     0.00                                                        280
    25    99.50     0.00 SUPPORT                                                290
JOINT RELEASES MOMENT Z                                                         300
     1                                                                          310
    25 FORCE X                                                                  320
MEMBER RELEASES START MOMENT Z END MOMENT Z                                     330
       26 27 28 29 30 31                                                        340
MEMBER INCIDENCES                                                               350
   1    1    2                                                                  360
   2    2    4                                                                  370
   3    4    5                                                                  380
   4    5    7                                                                  390
   5    7    8                                                                  400
   6    8   10                                                                  410
   7   10   11                                                                  420
   8   11   13                                                                  430
   9   13   14                                                                  440
  10   14   16                                                                  450
  11   16   17                                                                  460
```

S/C 2470

VERIFY

STATE OF CALIFORNIA - BUSINESS AND TRANSPORTATION AGENCY - DEPARTMENT OF PUBLIC WORKS - DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b		b	ADDRESS DIST.	GROUP	BATCH
	$		14	5	03

SUBSYSTEM NAME								b		b	SOURCE DIST. / UNIT	CHARGE DIST. / UNIT	EXPENDITURE AUTHORIZATION	SPECIAL DESIGNATION WHEN APPLICABLE	b	SEQUENCE
									$							ØØØ1

1-5	6-10	11-15	16-20	21-25	26-30	31-35	36-40	73-76
12	17	19						470
13	19	20						480
14	20	22						490
15	22	23						500
16	23	25						510
17	1	3						520
18	3	6						530
19	6	9						540
20	9	12						550
21	12	15						560
22	15	18						570
23	18	21						580
24	21	24						590
25	24	25						600
26	6	5						610
27	9	8						620
28	12	11						630
29	15	14						640
30	18	17						650
31	21	20						660
MEMBER PROPERTIES								670
	1 TO	16 PRISMATIC AX			6.75	IZ	5.06	680
	17 TO	25 PRISMATIC AX			10.08	IZ	23.15	690

s/c 2470

BY:
CHECK:
DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:
NAME G.W. Helker
PHONE 5-5408 DATE 1-12-72

VERIFY

PAGE 3 OF 4

HCS-329 (REV. 1/1/70)

ADDRESS DIST.	GROUP	BATCH
14	S	05

```
   26 TO 31 PRISMATIC AX  2.09 IZ  0.49                                   700
SUPERSTRUCTURE N 1 17 N 1 18 N 1 19 N 1 20 N 1 21 N 1 22 -               710
     N 1 23 N 1 24 N 1 25                                                  720
MOVE LOAD BOTH TRUCK NP 4 26 15.75 24 4.5 24 13.50 24 4.5 -               730
     24 13.50 24 4.5 24                                                    740
GENERATE LOADS Y SCALE -1.0 INITIAL 10 PRINT                              750
CONSTANT E 432000. ALL                                                    760
OUTPUT DECIMAL 3                                                          770
PRINT DATA                                                                780
STIFFNESS ANALYSIS                                                        790
LIST FORCE ENVELOPE ALL MEMBERS SECTION FRACTIONAL DS 0.0 1.0             800
```

S/C 2470

VERIFY

STATE OF CALIFORNIA – BUSINESS AND TRANSPORTATION AGENCY – DEPARTMENT OF PUBLIC WORKS – DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b		b	ADDRESS DIST.	ADDRESS GROUP	BATCH
	$		145		01

SUBSYSTEM NAME	b		b	SOURCE DIST.	SOURCE UNIT	CHARGE DIST.	CHARGE UNIT	EXPENDITURE AUTHORIZATION	SPECIAL DESIGNATION WHEN APPLICABLE	b	SEQUENCE
		$		146	02	146	01	9617014	23C092		ØØØ1

```
STRUDL 'STEVEN'  'BRIDGE 23C 92 10 SOL FAS L112'                          10
TYPE PLANE FRAME                                                          20
UNITS KIP FEET                                                            30
JOINT COORDINATES                                                         40
      1 X   0.00 Y  0.00 SUPPORT                                          50
      2     7.75    6.60                                                  60
      3     7.75    0.00                                                  70
      4    13.75   10.90                                                  80
      5    19.75   14.00                                                  90
      6    19.75    0.00                                                 100
      7    25.75   16.60                                                 110
      8    31.75   18.45                                                 120
      9    31.75    0.00                                                 130
     10    37.75   19.80                                                 140
     11    43.75   20.60                                                 150
     12    43.75    0.00                                                 160
     13    49.75   20.90                                                 170
     14    55.75   20.60                                                 180
     15    55.75    0.00                                                 190
     16    61.75   19.80                                                 200
     17    67.75   18.45                                                 210
     18    67.75    0.00                                                 220
     19    73.75   16.60                                                 230
```

S/C 2470

BY:
CHECK:
DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:
NAME George W. Heller
PHONE 5-5408 DATE 1-11-72

VERIFY

PAGE 1 OF 4

HCS-329 (REV. 1/1/70)

ADDRESS DIST. GROUP: 145 BATCH: 01

SEQUENCE CONTINUED

```
     20     79.75    14.00                     240
     21     79.75     0.00                     250
     22     85.75    10.90                     260
     23     91.75     6.60                     270
     24     91.75     0.00                     280
     25     99.50     0.00 SUPPORT             290
JOINT RELEASES MOMENT Z                        300
     1                                         310
     25 FORCE X                                320
MEMBER RELEASES START MOMENT Z END MOMENT Z    330
       26 27 28 29 30 31                       340
MEMBER INCIDENCES                              350
   1    1    2                                 360
   2    2    4                                 370
   3    4    5                                 380
   4    5    7                                 390
   5    7    8                                 400
   6    8   10                                 410
   7   10   11                                 420
   8   11   13                                 430
   9   13   14                                 440
  10   14   16                                 450
  11   16   17                                 460
```

S/C 2470

VERIFY

STATE OF CALIFORNIA – BUSINESS AND TRANSPORTATION AGENCY – DEPARTMENT OF PUBLIC WORKS – DIVISION OF ADMINISTRATIVE SERVICES

COMPUTER SYSTEMS

ICES

b	$	b	ADDRESS DIST.	GROUP	BATCH
	$		143		01

SUBSYSTEM NAME (1–5)	6–10	11–15	16–20	21–25	26–30	31–35	36–40	41–63	SEQUENCE
									ØØØ1
12	17	19							470
13	19	20							480
14	20	22							490
15	22	23							500
16	23	25							510
17	1	3							520
18	3	6							530
19	6	9							540
20	9	12							550
21	12	15							560
22	15	18							570
23	18	21							580
24	21	24							590
25	24	25							600
26	6	5							610
27	9	8							620
28	12	11							630
29	15	14							640
30	18	17							650
31	21	20							660
MEMBER	PROPERTIES								670
	1 TO	16 PRISMATIC		AX	6.75	IZ	5.06		680
	17 TO	25 PRISMATIC		AX	10.08	IZ	23.15		690

BY:
CHECK:
DATE:

REMARKS:

IN CASE OF QUESTION CONTACT:
NAME GW Holler
PHONE DATE 1-11-72

VERIFY

S/C 2470

PAGE 3 OF 4

HCS-329 (REV. 1/1/70)

ADDRESS DIST. GROUP BATCH: 145 01

SEQUENCE CONTINUED

```
    26 TO 31 PRISMATIC AX 2.09 IZ 0.49                                  700
LOADING 1 'UNIFORM LOADS DEAD LOAD OF MEMBER'                           710
     MEM 1 TO 16 LOAD FORCE Y GLOBAL UNI W -1.01                        720
     MEM 17 TO 25 LOAD FORCE Y UNI W -3.29                              730
     MEM 26 TO 31 LOAD FORCE Y GLOBAL UNI W -0.31                       740
LOADING 2 'CONCENTRATED STRUT DEAD LOAD AT JOINT'                       750
     JOINT 8 LOAD FORCE Y -5.64                                         760
     JOINT 17 LOAD FORCE Y -5.64                                        770
LOADING COMBINATION 3 'DEAD LOADS' COMBINE 1 1.0 2 1.0                  780
SUPERSTRUCTURE N 1 17 N 1 18 N 1 19 N 1 20 N 1 21 N 1 22 -              790
     N 1 23 N 1 24 N 1 25                                               800
MOVE LOAD BOTH TRUCK NP 3 8 11 16 4 16 22 16 4 16                       810
GENERATE LOADS Y SCALE -1.0 INITIAL 10 PRINT                            820
CONSTANT E 432000. ALL                                                  830
OUTPUT DECIMAL 3                                                        840
PRINT DATA                                                              850
STIFFNESS ANALYSIS                                                      860
LOAD LIST 1 2 3                                                         870
LIST FORCES LOADS REACTIONS DISPLACEMENTS                               880
LOAD LIST ALL BUT 1 2 3                                                 890
LIST FORCE ENVELOPE ALL MEMBERS SECTION FRACTIONAL DS 0.0 1.0           900
```

S/C 2470

VERIFY

BRIDGE No. 23C-92
SHEET
DATE 9-15-71

DIVISION OF HIGHWAYS
DELIVER-BRIDGE DEPT.
SACRAMENTO

1971 SEP 16 PM 3 17

BRIDGE
A.S.R. ENG'R.
ADMIN.
OFFICE ENG'R.
OPERATIONS
ADV. PLAN
DESIGN
MAINTENANCE
ACCOUNTING
FED. AID SEC.
AGREEMENTS
PERSONNEL
RESEARCH
GEOLOGY
STENOS
SHOP PLANS
SPEC. & EST.

To: George Hood

Attached is the plan sheet for Br. No. 23C-92, an arch structure. Please have this structure computer analyzed for legal and overload conditions.

Arch ribs seem to be in fair condition and may probably be rated at $f_c = 800$ psi

But girders and decks are a problem.

In arch spans the deck has numerous transverse cracks which go completely through with extensive staining. Some cracks extend to and down exterior longitudinal girders. Some soffit rebar is exposed

In approach spans (T-beam) the deck is in the same condition with cracks running down exterior girders also.

These cracks are fine line in most cases and are apparently due to reinforcement rusting combined with excessive deflections by heavy truck or farm vehicles.

I would estimate a 25% loss in reinforcement for the deck and girder systems and $f_c = 600$ psi.

Please return plan sheet when you return computer results.

Bernard S. LaPedis

— No CHECK — Calculated by GWH 2-72

... D M 18

EST. 3448. 26390-500 11-68 4M OSP

Sheet 1 of 3

STATE OF CALIFORNIA

BRIDGE REPORT

BRIDGE ALTERED SEE REVISED ORIGINAL REPORT

Bridge No. 23C-92

Date of Investigation July 9, 1971

Location 10-SOL-FAS 1112
Dist.-Co.-Rte. - City - P.M.

Name Putah Creek (Stevenson or Rumsey Bridge)

Location 6.3 miles N. of junction with FAI 80 on Stevenson Bridge Road

Latitude 38° 32.4' Longitude 121° 51.1' DOD Rd. ______ Section Br. ______ Letter

Custodian Solano and Yolo Counties Owner Solano and Yolo Counties

CLASSIFICATION

Federal Aid System 07 Administrative 4 Functional 8 5

STRUCTURAL DATA AND HISTORY

Year Built 1923 By Solano & Yolo Counties Cont. No. Unknown FAP No. ______

Designed by Counties Plans Avail Yes at State Bridge Dept.

Spans 1 @ 40.0' - 2 @ 108.0', 1 @ 40.0' c/c Length 298' Skew None

Description 2 RC Tied arch center spans with RC girder (5) approach span each side, RC 2-column piers on RC piles, and RC wing abutments on spread footings.

On Structure

Roadway Section 2 @ 10.0' Total Width 24.2'

Rail Type Lt. RC Rt. RC Median None Lanes 2 No. Tracks N/A No.

Clearances: Vert. 14'-3" Horiz. 20.0' PUC No. N/A

Design LL County-medium Overload Rating Legal loads only

Under Structure

Roadway Section N/A Lanes N/A No. Tracks N/A No.

Clearances: Vert N/A Horiz. N/A PUC No. N/A

HYDRAULIC STRUCTURE

Report? Yes ☐ No ☒ Nav. Control Yes ☐ No ☒ Clear. Diag. Yes ☒ No. ☐

Relief Structures None

APPRAISAL OF NON-STRUCTURAL FACTORS

Deck Geometry 4 Approach Alignment 3

Waterway Adequacy 8 Clearances (Vert. & Horiz.) 4 & 4

Bridge No. 23C-92

STATE OF CALIFORNIA

BRIDGE REPORT

Date of Investigation July 9, 1971

ADT 284 Year 1970

RATING OF CONDITION:

Deck 4 Substructure 5

Superstructure 6 Overall 4

PLANS AND DIMENSIONS

General construction plans are available at the State Bridge Department.

EXISTING POSTING

There is no record of any restrictive posting for load or speed by the Director but there are the following signs posted due to the bridge's geometry. A 15mph sign is posted facing approaching downlog traffic due to the 90° turn at the downlog end of the bridge; and 14'-3" vertical clearance signs are posted in the first portal at each end, facing approaching traffic.

CONDITION OF STRUCTURE

Deck slab in each span has numerous transverse cracks, both on top and on soffit. Some of these cracks extend down exterior girders. Deck soffit reinforcement is exposed at a few locations and rust and efflorescent stains are visible. It is estimated that there is some section loss of reinforcement, but it is minor at this time.

At A5 there is a slight separation at the wingwall joints. At Pier 2, there is a wing retaining wall projecting from left column. This wall has completely separated from column.

Soundings around Pier 3 indicate that scour is occurring around piles, exposing approximately 3 feet of piles below footing block.

WATERWAY

Channel has a sand and gravel bottom and appears to be sufficient.

ENCROACHMENTS

At Pier 3 there is a 3 foot diameter CMP stilling well along the right side of the pier. Stilling well extends from railing height to channel bottom. It is supported at intervals from column and footing.

CAPACITY

Bridge is good for legal loads and for no overloads as determined by computer analysis by headquarters personnel.

POSTING RECOMMENDATION

None.

Bernard S. LaPedis

Bernard S. LaPedis
Associate Bridge Engineer
P. E. Lic. No. C-17067

EST. 613. 57774-800 11-71 3M OSP

DRAWN BY TP DATE 7-9-71

CHECKED BY BSL

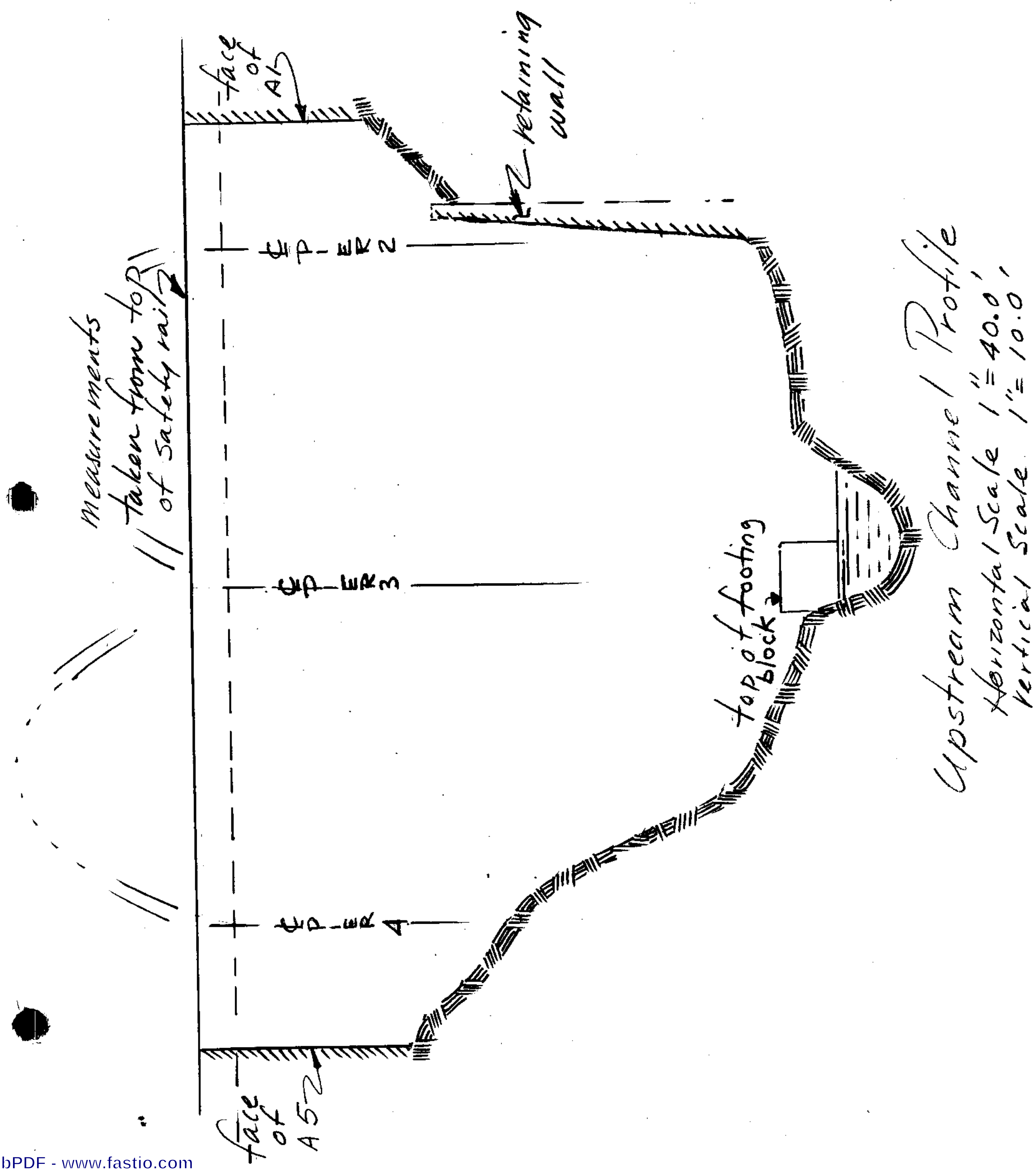
APPROVED

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
BRIDGE DEPARTMENT

DIST.	COUNTY	ROUTE	POST MILE
10	SOL	FAS 1112	

NAME Putah Creek

BRIDGE NO. 23C-92



Appendix J - Meeting Minutes

Stevenson Bridge Road
Bridge Project

Kick-off Meeting Minutes
May 25, 2016 2:00 pm to 4:00 pm

675 Texas Street, Suite 5500
Fairfield, CA 94533
Minutes by Lance Schrey of Quincy Engineering Inc.

1) Introductions

In attendance were the following:

Nick Burton	Solano County
Nathan Newell	Solano County
Bob Liu	Solano County
John Quincy	Quincy Engineering Inc.
Lance Schrey	Quincy Engineering Inc.
Jason Chou	Quincy Engineering Inc.
Reimond Garcia	Quincy Engineering Inc.
Frank Cannizzaro	Alta Vista
Jinesh Mehta	Alta Vista
Chris Hockett	Cal Engineering & Geology
Rocio Briseno	Cal Engineering & Geology
Han-Bin Liang	WRECO

Sign in sheet is attached

2) Project Areas

a) Existing Bridge

Lance noted the following:

i) The existing bridge is approximately 298 feet long by 23 feet wide and is almost 100 years old.
ii) The bridge is supported by spread footings at the abutments and piles at the piers.
iii) The latest Caltrans inspection report notes the following conditions:
  (1) Numerous spalls with exposed rebar.
  (2) Transverse soffit cracks in the end spans.
  (3) 40% of girders have spalls.
  (4) 50% of the arch have spalls or delamination.
  (5) Pier 3 has 58 inches of exposed pile cap.
iv) Previously the County had a consultant prepare a Feasibility Study Report.
  (1) The report found the existing structure O.K. for legal loads.
  (2) Under seismic conditions the report noted several elements exceed their capacity.

(3) The report investigated two rehabilitation options and one replacement option.

b) Surveying

i) The County will provide Quincy with past survey information.

ii) The County will acquire any needed additional survey information.

iii) Surveyors will not need right of entry.

iv) The County will obtain rights of entry for geotechnical work to be performed.

c) Hydraulics

i) Han noted the need for 6 to 8 additional cross sections. After receiving existing survey data, Han will mark locations for additional cross sections.

ii) Han noted back in 2006 this site was not under the jurisdiction of the Central Valley Flood Protection Board (CVFPB). He will revisit.

iii) The County noted that since this project lies within the Federal Project Zone, the CVFPB permit (with co-review by the Army Corps of Engineers) will take between 9 and 12 months to acquire.

d) Geotechnical

i) There is boring data from both abutments from the previous prepared Foundation Report prepared by Kleinfelder. Chris requested the additional appendices from this report.

ii) Chris noted that they were scoped to perform one boring at the center pier. They anticipate craning a track mounted rig from bridge to the drilling location.

iii) Bob noted that he believed FHWA will require a boring at each foundation location. Also, the County has concerns of the Contractor filing a claim for differing site conditions if there is not a boring at each support. Lance will investigate the need for additional borings. Chris to develop a scope and fee for two additional borings at the outside piers.

iv) Nick does not believe a Fish and Wildlife Permit will be required to perform the drilling. Nathan will investigate.

v) Permits from both Solano County and Yolo County are required for drilling two additional borings at the outside piers.

vi) Chris noted the additional information that could be provided and possible project cost savings if sCPT's are performed. Currently the contract has scoped two sCPT's as optional tasks. Chris handed out a figure (attached) showing different ARS Curves associated with different shear wave velocities. The County to determine if the optional borings are to be performed.

vii) Chris noted he did not anticipate a high probability of liquefaction at the site.

viii) The County will help with coordination with PG&E for drilling.

e) Structure Assessment

QUINCY ENGINEERING

i) The County will provide Quincy with past mapping and structural assessment information.
ii) Bob informed the team of a job (Laurel Street Bridge Overcrossing) where similar work was performed. Lance will obtain information on this job to use as an example.
iii) Alta Vista would like to use a drone to preliminarily assess the structure.
  (1) Alta Vista said they could do this within the budget they provided.
  (2) The County is O.K. with using a drone as long as it is approved by Caltrans. Nathan noted the Caltrans District 4 Architectural Historian is Helen Blackmore and that all communications with her should go through the County.

f) Preliminary Engineering

i) Roadway
  (1) Reimond handed out an alignment exhibit and discussed alternative alignments.
  (2) The County discussed a Farmland Memo. They will provide a copy to Quincy.
  (3) Being conscious of the high volume of cyclist traffic, the County prefers to change the County standard 4 foot unpaved shoulders to four foot paved shoulders.
  (4) The County to provide the Caltrans' response on the preferred alignment.
  (5) The County to provide the Traffic Index.
  (6) The County shared that Caltrans prefers a lower design speed alternative.
  (7) The County prefers to have the roadway designed to standards with superelevation if possible.
  (8) A discussion on Functional Obsolete and recent Caltrans funding change. The project is already programed so it is not effected by the change.

ii) Access Road
  (1) The County concurred with how the access road and temporary water crossing is depicted in the alignment exhibit.

iii) Structure Modeling
  (1) Jason discussed the modeling of the neighboring Rumsey Bridge in Yolo County and modeling of the Stevenson Bridge Road Bridge.

iv) Utility Coordination
  (1) The County will provide Quincy with a utility list. Quincy will then send out "A" letters.

g) Environmental
i) NES and BA have been approved. The County will provide Quincy copies.

ii) The County would prefer not to go to bid until all permits are obtained.
iii) Nathan noted that there are approximately 20 Elderberry Bushes in the area.

h) Public Outreach

i) The County wants to initiate Public Outreach when the 30% plans are completed.

3) Project Schedule

a) Lance handed out a schedule. He noted that this was just a starting point and wanted everyone to review and get him feedback so he can update the schedule. Attached is the revised schedule.

4) Site Visit

a) Upon completion of the meeting the team performed a site visit.
   i) Nathan pointed out if the additional borings are performed at the outside piers, it would be beneficial to alternate the sides of the piers where the borings are performed.
   ii) Bird nests were noted on the bridge. Geotechnical investigations will not be able to start until after the nesting birds on the bridge have fledged.
   iii) Two bat boxes were observed on the underside of the bridge.
   iv) Chris noted the location he proposes to lower the track mounted crane to perform the boring of the center pier. It was noted that a walnut tree will need to be trimmed or removed. Nathan will look into this.\
   v) The County noted that they should be able to perform tree pruning so that assessment and bridge inspection can be performed without any obstructions.

5) Post Meeting

a) Since there is some discrepancy between the as-built plans and normal convention for support labeling, the County wants the following convention to be used for the duration of the project:
   i) Abutment 1 – Southern Abutment
   ii) Pier 2 - Southern Pier
   iii) Pier 3 – Middle Pier
   iv) Pier 4 – Northern Pier
   v) Abutment 5 – Northern Abutment

6) Action Items

	Item	Who	Status
1	Survey information	County	**Completed** – The County sent file on 5/26/2016.
2	Rights of entry	County	**Pending -**
3	Locations needed for additional survey cross sections.	WRECO	**Completed** – Quincy sent survey request to the County on 6/7/2016.
4	Appendix from original Foundation Report.	County	**Completed** – The County sent files on 6/3/2016.
5	Scope and fee for additional borings	Cal Eng. & Geology	**Completed** – Quincy sent scope and fee to the County on 6/7/2016. The scope and fee was revised and resent on 6/9/2016.
6	The need for a Fish and Wildlife Permit to perform boring.	County	**Pending -**
7	Past mapping and structural assessment information.	County	**Pending -**
8	Information on the requirement for additional borings.	Quincy	**Completed** – Quincy provided information to the County on 6/7/2016.
9	Laurel Street Bridge Overcrossing information.	Quincy	**Completed** – Quincy sent files to the County on 6/1/2016.
10	Farmland Memo	County	**Pending -**
11	Caltrans preferred alignment	County	**Pending -**
12	Traffic Index	County	**Pending -**
13	Utility List	County	**Pending -**
14	Send out Utility "A" letters	Quincy	**Pending -**
15	Copies of approved NES and BA	County	**Completed** – The County sent files on 5/26/2016.

Attachments:

Sign in sheet
Meeting Agenda
Potential ARS Curve Figure
Alignment Exhibit
Project Schedule

STEVENSON BRIDGE ROAD BRIDGE

Kick-off Meeting – May 25, 2016: 2:00 p.m. – 4:00pm

Name	Organization	Telephone	Email address
Lance Schrey	Quincy	916-368-9181	Lances@quincyeg.com
Jason Chou	Quincy	916-368-9181	Jchou@quincyeng.com
Reimond Garcia	Quincy	916-368-9181	Reimondg@quincyeng.com
John Quincy	QEI	" " "	Johnq@quincyeng.com.
Nathan Newell	Solano Co.	707-784-3095	nlnewell@solanocounty.com
Nick Burton	Solano Co	707 784 3155	nsburton@solanocounty.com
Frank Cannizzaro	Alta Vista	510-326-0453	fcannizzaro@altavistasolutions.com
ROCIO BRISENO	Cal Engineering & Geology	(510)229-0899	rbriseno@caleng.com
CHRIS HOCKETT	CAL ENGINEERING & GEOLOGY	925.935.9771	chockett@caleng.com
Jinesh Mehta	Alta Vista	951 560-0321	jmehta@altavistasolutions.com
Han-Bin Liang	WRECO	(925) 941-0017	HanBin_Liang@wreco.com
Robert Liu	Solano County	(707) 784-6074	rliu@Solanocounty.com

Stevenson Bridge Road
Bridge Project

Kick-off Meeting Agenda
May 25, 2016   2:00 pm to 4:00 pm

675 Texas Street, Suite 5500
Fairfield, CA 94533

I. Introductions (Sign In Sheet)

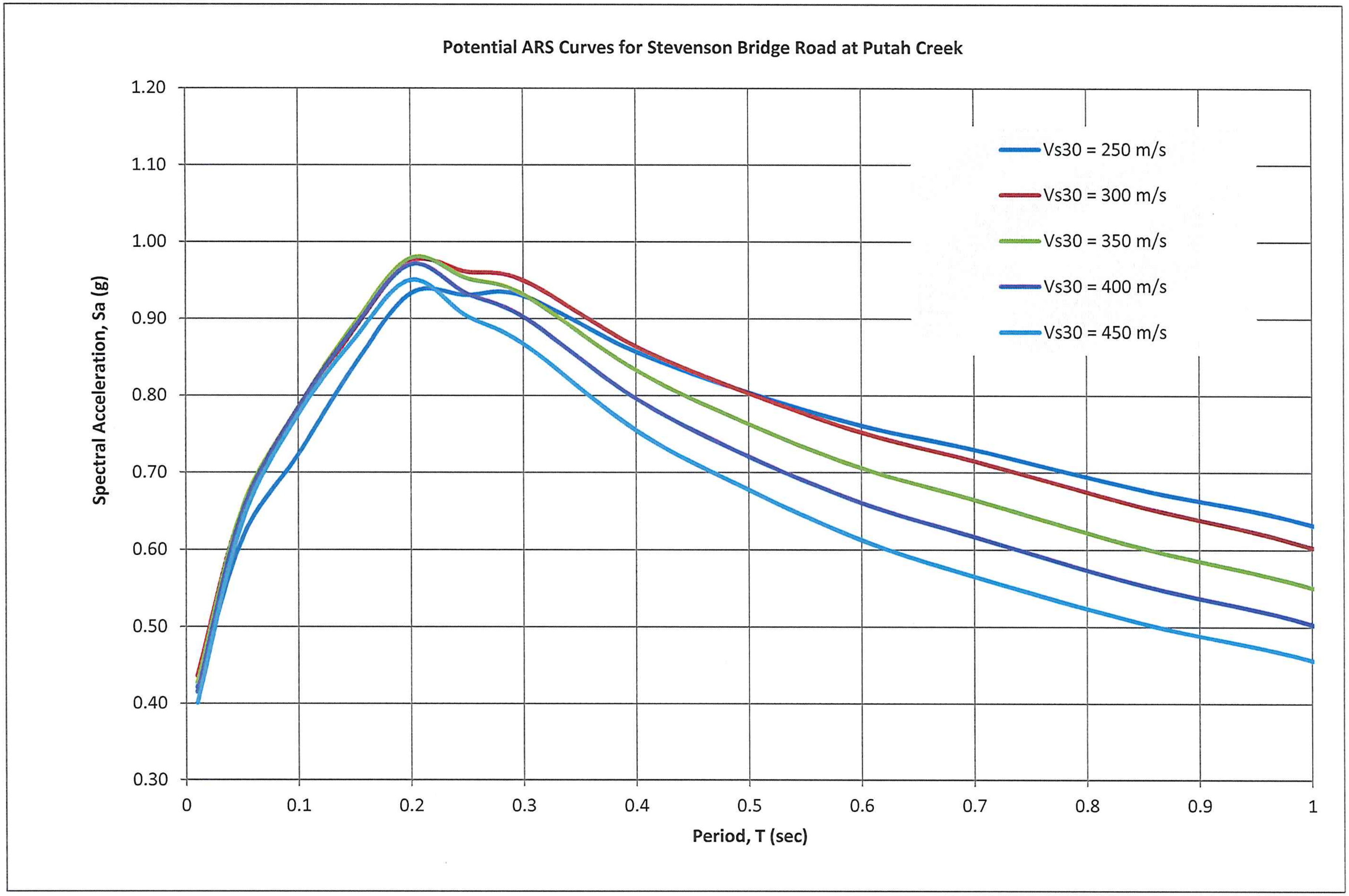
II. Project Areas

   a. Existing Bridge (Quincy)
      i. History, As-built Plans & Inspection Reports
   b. Surveying (County)
      i. Record research/Parcels affected
      ii. Existing Survey Information
   c. Hydraulics (WRECO)
      i. Existing Report
      ii. Report Update
   d. Geotechnical (Cal Engineering and Geology)
      i. Right of entry
      ii. Encroachment permits (both counties)
      iii. Fish & Wildlife Permit
      iv. Existing Information (past fnd. report & well reports)
      v. Borings
         1. Center Pier
         2. Abutment sCPT's (Optional Task)
   e. Structure Assessment (Alta Vista)
      i. Visual Assessment
      ii. Mapping Diagram
      iii. Risk Based Investigation
      iv. Strength Testing
   f. Preliminary Engineering (Quincy)
      i. Basis of Design
         1. Design Speed
         2. Roadway Alignments
      ii. Access Road
      iii. Structure Modeling
      iv. Utility Coordination
   g. Environmental (County)
   h. Public Outreach (County & Quincy)

III. Project Schedule (Quincy)

IV. Site Visit (All)

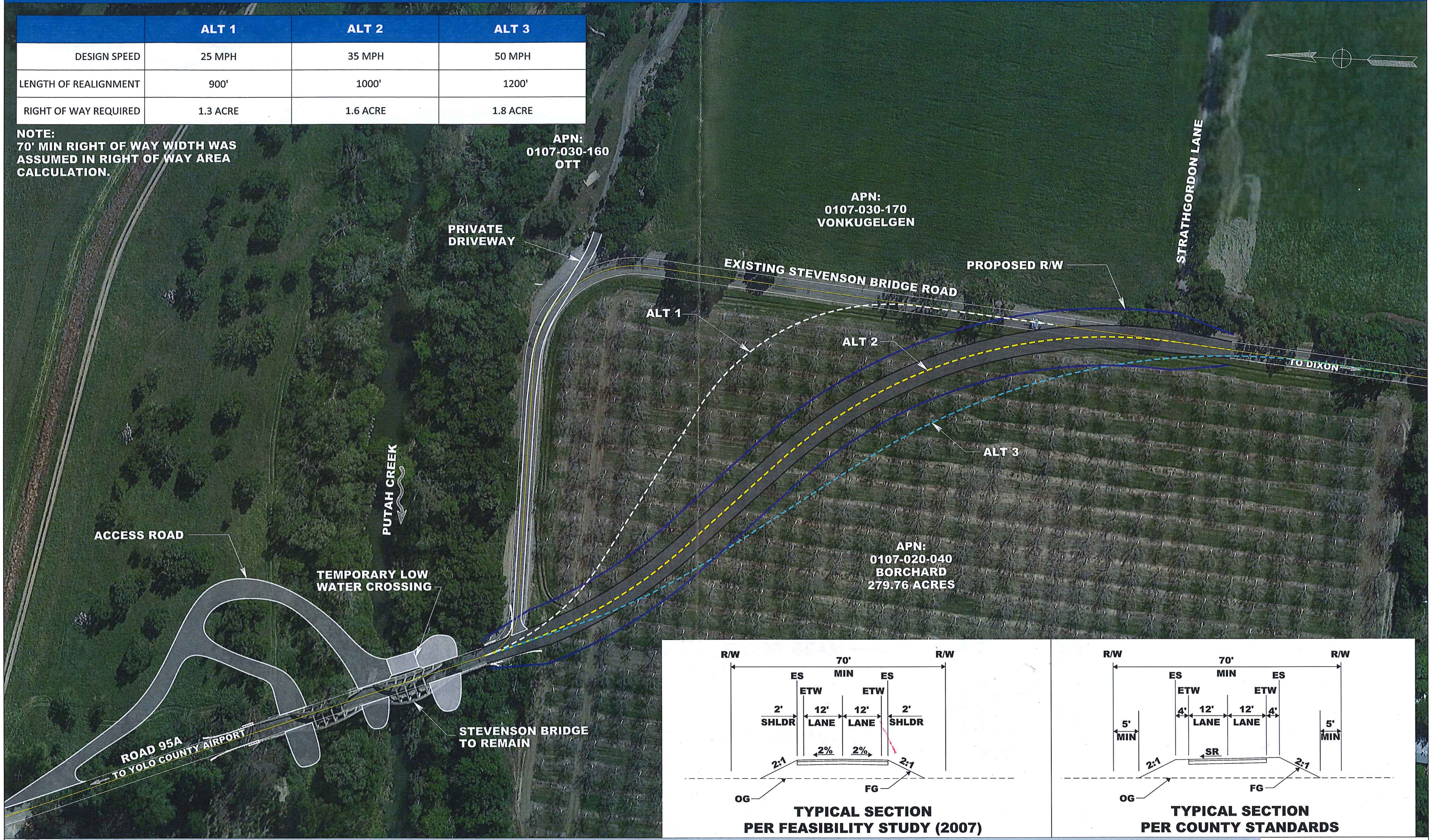
Cal Engineering & Geology



5/25/2016

	ALT 1	ALT 2	ALT 3
DESIGN SPEED	25 MPH	35 MPH	50 MPH
LENGTH OF REALIGNMENT	900'	1000'	1200'
RIGHT OF WAY REQUIRED	1.3 ACRE	1.6 ACRE	1.8 ACRE

NOTE:
70' MIN RIGHT OF WAY WIDTH WAS ASSUMED IN RIGHT OF WAY AREA CALCULATION.



SOLANO COUNTY - DEPARTMENT OF RESOURCE MANAGEMENT

STEVENSON BRIDGE ROAD REHABILITION PROJECT

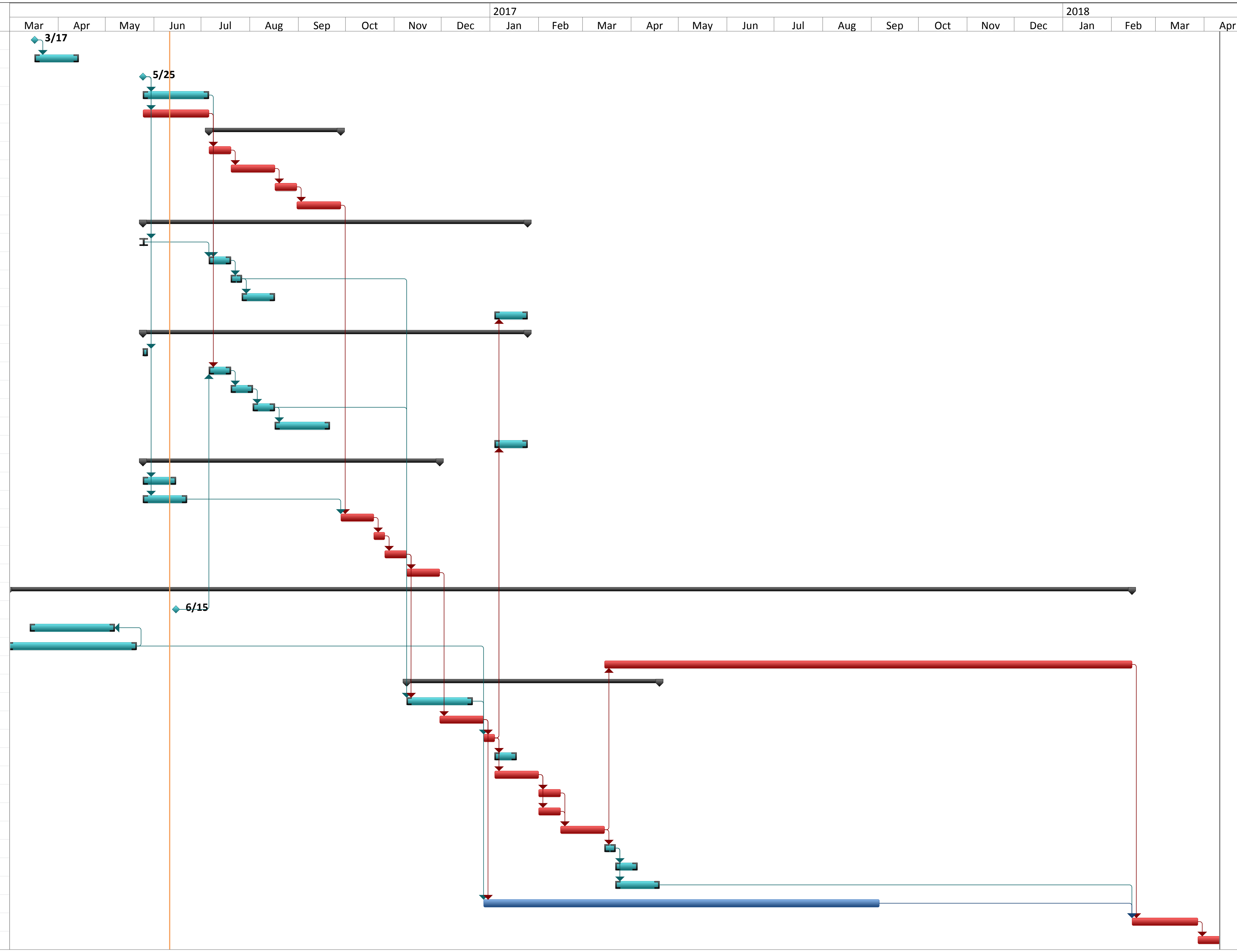
ROADWAY ALIGNMENT ALTERNATIVES

EXHIBIT 2

QUINCY ENGINEERING

SOLANO COUNTY - STEVENSON BRIDGE ROAD BRIDGE REHABILITATION SCHEDULE

ID	Task Name	Duration	Start	Finish	Predecessors
1	Execution of Agreement (NTP)	0 days	Thu 3/17/16	Thu 3/17/16	
2	Subconsultant Agreements	4 wks	Thu 3/17/16	Wed 4/13/16	1
3	Kick-off Meeting	0 days	Wed 5/25/16	Wed 5/25/16	
4	Updated Survey Info.	6 wks	Wed 5/25/16	Tue 7/5/16	3
5	Right of Entry	6 wks	Wed 5/25/16	Tue 7/5/16	3
6	**Structure Assessment**	**60 days**	**Wed 7/6/16**	**Tue 9/27/16**	
7	Visual Inspection	2 wks	Wed 7/6/16	Tue 7/19/16	5
8	Risk Based Investigation	4 wks	Wed 7/20/16	Tue 8/16/16	7
9	Strength Testing	2 wks	Wed 8/17/16	Tue 8/30/16	8
10	Assessment Report	4 wks	Wed 8/31/16	Tue 9/27/16	9
11	**Hydraulics**	**175 days**	**Wed 5/25/16**	**Tue 1/24/17**	
12	Site Recon.	1 day	Wed 5/25/16	Wed 5/25/16	3
13	Hydraulic Analysis	2 wks	Wed 7/6/16	Tue 7/19/16	12,4
14	Scour Analysis	1 wk	Wed 7/20/16	Tue 7/26/16	13
15	Draft Hydraulic Study Report	3 wks	Wed 7/27/16	Tue 8/16/16	14
16	Final Hydraulic Study Report	3 wks	Wed 1/4/17	Tue 1/24/17	39
17	**Geotechnical**	**175 days**	**Wed 5/25/16**	**Tue 1/24/17**	
18	Recon. / Permitting	3 days	Wed 5/25/16	Fri 5/27/16	3
19	Subsurface Exploration	2 wks	Wed 7/6/16	Tue 7/19/16	32,5
20	Lab Testing	2 wks	Wed 7/20/16	Tue 8/2/16	19
21	Geotechnical Analysis	2 wks	Wed 8/3/16	Tue 8/16/16	20
22	Draft Foundation Report	5 wks	Wed 8/17/16	Tue 9/20/16	21
23	Final Foundation Report	3 wks	Wed 1/4/17	Tue 1/24/17	39
24	**Preliminary Engineering**	**135 days**	**Wed 5/25/16**	**Tue 11/29/16**	
25	Basis of design	3 wks	Wed 5/25/16	Tue 6/14/16	3
26	Structure Modeling	4 wks	Wed 5/25/16	Tue 6/21/16	3
27	Retrofit Strategy	3 wks	Wed 9/28/16	Tue 10/18/16	26,10
28	Project Report	1 wk	Ved 10/19/16	Tue 10/25/16	27
29	County Review	2 wks	Ved 10/26/16	Tue 11/8/16	28
30	30% Roadway Design	3 wks	Wed 11/9/16	Tue 11/29/16	29
31	**Environmental Clearance**	**512 days**	**Mon 2/29/16**	**Tue 2/13/18**	
32	Swallows Leave	0 days	Wed 6/15/16	Wed 6/15/16	
33	Environmantal Assistance	2 mons	Mon 3/14/16	Fri 5/6/16	34FF-2 wks
34	NEP / CEQA	3 mons	Mon 2/29/16	Fri 5/20/16	
35	Permitting	12 mons	Wed 3/15/17	Tue 2/13/18	44
36	**PS&E**	**115 days**	**Wed 11/9/16**	**Tue 4/18/17**	
37	Bridge Design	6 wks	Wed 11/9/16	Tue 12/20/16	29,14,21
38	Roadway Design	1 mon	Ved 11/30/16	Tue 12/27/16	30
39	50% Plans	1 wk	Ved 12/28/16	Tue 1/3/17	37,38
40	County Review	2 wks	Wed 1/4/17	Tue 1/17/17	39
41	Independent Design Check	4 wks	Wed 1/4/17	Tue 1/31/17	39
42	Special Provisions	2 wks	Wed 2/1/17	Tue 2/14/17	41
43	Quantities /Estimate	2 wks	Wed 2/1/17	Tue 2/14/17	41
44	90 % PS&E	4 wks	Wed 2/15/17	Tue 3/14/17	42,43
45	QA / QC	1 wk	Wed 3/15/17	Tue 3/21/17	44
46	County Review	2 wks	Wed 3/22/17	Tue 4/4/17	45
47	Final PS&E	4 wks	Wed 3/22/17	Tue 4/18/17	45
48	Right of Way Acquisition	9 mons	Ved 12/28/16	Tue 9/5/17	34,38
49	Advertise	6 wks	Wed 2/14/18	Tue 3/27/18	47,48,35
50	Construction	2 wks	Wed 3/28/18	Tue 4/10/18	49



Project: Stevenson Schedule rev3
Date: Sat 6/11/16

Task	Summary	External Milestone	Inactive Summary	Manual Summary Rollup	Finish-only	Critical Split	
Split	Project Summary	Inactive Task	Manual Task	Manual Summary	Deadline	Progress	
Milestone	External Tasks	Inactive Milestone	Duration-only	Start-only	Critical		

Stevenson Bridge Road Bridge Project

Structure Assessment Meeting Minutes
October 5, 2016 10:45 am to 12:30pm

675 Texas Street, Suite 5500
Fairfield, CA 94533
Minutes by Lance Schrey of Quincy Engineering Inc.

I. Introductions

In attendance were the following:

Nathan Newell	Solano County
Lance Schrey	Quincy Engineering Inc.
Jason Chou	Quincy Engineering Inc.
Jinesh Mehta	Alta Vista
Aaron Prchlik	Alta Vista

Sign in sheet is attached

II. Project Update

a. Addendum 1

Lance gave Nathan two copies of Addendum 1 (for the additional borings) for signature.

b. Geotechnical

i. Lance gave Nathan a copy of the boring logs for Borings B1 and B2.
ii. Lance noted that boring B3 will take place on October 18^{th} & 19^{th}.

c. Hydraulics

i. Lance noted that WRECO is working on the hydraulics and they have determined that there is a discrepancy between flows calculated and gauge readings. It appears that the scour amounts previously calculated may be conservative.
ii. Lance noted that WRECO is planning on contacting the Central Valley Flood Protection Board to determine what they will require for this project.

d. Structure Model

Jason updated the team on the state of his models including showing plots of the extruded 3D view of the model, and identifying locations of seismic vulnerabilities.

III. Structure Assessment

a. Jinesh distributed a meeting handout (attached).
b. Jinesh and Aaron went over the results of the visual assessment.
    i. This included a mapping plan showing the locations of the highest damaged areas.
    ii. The worst damage is in spans 1 and 4 where there are cracks in the ¾ span location. These cracks start in the deck and propagate to within several inches of the bottom of the girders.
        1. Based on previous reports, these cracks could be more than 10 years old.
        2. The cause of the cracks may be due to abutment settlement, and/or loads from the arch span causing excessive negative moments on the approach span.
        3. It was noted that it appears that there is no top mat reinforcements in girders in span 1 & 4.
        4. It is unclear if the shear reinforcement is full length for the girders in spans 1 & 4. GRP will be used to verify the extent of shear reinforcement.
        5. Reinforcing steel at the cracked locations may be corroded. Risk based investigation will used to verify the extent of the corrosion.

c. Included in the aforementioned meeting handout (attached) was a table listing several options for the risk based investigation.
    i. This included work that was outside of Alta Vista's original scope of work.
    ii. Since at this time it is not clear where additional information is needed, it was agreed to just perform the scoped work.
        1. This includes 4 cores to determine concrete strength.
        2. It was agreed that 2 of the cores would be at the major crack location to determine more information in that area.
        3. A third core will be taken in the arch near the spring line.
        4. Quincy will inform Alta Vista where to take the 4th coring at a later time.

5. Alta Vista is planning on performing risk based investigation in early November

d. Prior to performing the field work, Alta Vista will provide locations of the risk based testing (GPR, borescope, and strength cores).

e. To insure that all project information follows the same convention, Nathan noted everyone should follow the normal Caltrans convention as noted below.
    - i) Abutment 1 – Southern Abutment
    - ii) Pier 2 - Southern Pier
    - iii) Pier 3 – Middle Pier
    - iv) Pier 4 – Northern Pier
    - v) Abutment 5 – Northern Abutment

IV. Project Schedule

a. Lance noted that the schedule had been updated with 2 changes (copy is attached)
    - i. A line was added for the Structure Assessment Meeting.
    - ii. Time was added for the Subsurface Exploration.
    - iii. The revised schedule, with 12 months for Permitting and 9 months for Right of Way Acquisition, has construction beginning in June of 2018.

Attachments:

Sign in sheet
Meeting Agenda
Alta Vista Meeting Handout
Updated Schedule

Stevenson Bridge Road Bridge Project

Structure Assessment Meeting Agenda
October 5, 2016 10:00 am to 11:30 am

675 Texas Street, Suite 5500
Fairfield, CA 94533

I. Introductions (Sign In Sheet)

II. Project Update
- **a. Addendum 1 (extra borings)**
- **b. Geotechnical**
  - **i. Preliminary Results**
  - **ii. Final Boring**
- **c. Hydraulics**
  - **i. Preliminary Results**
- **d. Structure Model**

III. Structure Assessment
- **a. Visual Inspection Results**
- **b. Risk-based Investigation**

IV. Project Schedule

STEVENSON BRIDGE ROAD BRIDGE

Structure Assessment Meeting – Oct. 5, 2016: 10:00 am – 11:30 am

Name	Organization	Telephone	Email address
Lance Schrey	Quincy	916-368-9181	Lances@quincyeg.com
Jason Chou	Quincy	916-368-9181	Jchou@quincyeng.com
Jineel Mehta	Alta Vista	951-850-0531	jmehta@altavistasolutions.com
Aaron Prchlik	Alta Vista	510-610-9054	aprchlik@altavista solutions.com
Nathan Newell	Solano Co.	707.784.3095	nlnewell@solanocounty.com

October 5, 2016

Pre-Risk Based Investigation Meeting

Agenda:

1. Review of available documentation
2. What we found
3. What we are recommending and why
4. Schedule

1. Available documentation:

We reviewed all available data including previous studies, the Feasibility Report and the most recent Caltrans Bridge Inspection Reports.

- Reports indicate that the structure is considered "Structurally Deficient" and "Functionally Obsolete."
- Previously taken cores (20) indicate that the compressive strength in the structure (10+/- years ago) ranged between 1,920 psi and 3,470 psi.
- The retaining wall at the column of pier 2 has failed and detached from the structure
- Numerous defects including cracks in spalls in deck, girders, hanger columns and the arches

2. What we found:

The entire structure has been photographed. Photos of the abutments, wing walls, columns, interior girders, and deck soffit were photographed manually from the ground. Aerial photos of the deck, exterior girders, arches, hanger columns and railing were taken using an unmanned aerial vehicle (UAV). Based on our visual inspection and as shown in the photographs provided to the County we have observed the following:

- Both approach spans (spans 1 and 4) have significant structural defects including:
    - Major transverse cracks in the deck which extend down into the supporting girders as major vertical cracks in the girders. The cracks occur ¾ of the way into the span closer to the piers (further from the abutments).
- Spans 2 and 3 also show major signs of distress including:
    - Major spalls with exposed rebar in numerous bays and transverse floor beams.
- Several hanger columns have significant defects
- The arches have significant defects
- The columns don't show significant signs of defects on the exterior
- The abutments appear to have settled and cracked in the corners

October 5, 2016

Alta Vista will perform a Risk-Based Investigation based on the findings from our initial visual assessment as part of our scope of services.

Per our contract,

Task 2.1 – Reconnaissance/Visual Assessment

AVS will review available data, including previous studies, the Feasibility Report and all other pertinent information provided by the County as well as the most recent Caltrans Bridge Inspection Reports. **AVS** will perform a visual inspection of the bridge. From this assessment they will prepare a mapping diagram of areas requiring further testing. (Complete)

Task 2.2 – Risk Based Investigation

After completing the Mapping Diagram, **AVS** will meet with **Quincy** and the **County** to discuss what risk base investigation will be required. (Meeting scheduled for October 5, 2016). AVS assumes the follow tests will need to be performed as well as the frequency of each test.

- Ground Penetrating Rader – 80 locations for general condition assessment
- UPV testing – 40 locations for accurate deterioration depth determination

Propose to perform 120 GPR readings. 30%-40% deck, arches, columns. GPR readings scheduled to be taken on October 13th and 14th (if a second day is necessary)

- Borescope Inspection – 10 locations (validation of findings)

Borescope locations will be determined in the field while working in conjunction with GPR. Locations will be shown on mapping diagram. Drill and borescope work proposed to be performed on October 13th and 14th in conjunction with the GPR to confirm readings/findings.

Task 2.3 – Strength Testing

To augment the existing boring information AVS assumes that they will need to take an additional 4 reduced size cores to give an accurate determination of the concrete strength per ASTM C42.

We are proposing to take the cores on October 13th and 14th. Will cure, prepare and break the specimens in accordance with ASTM C42. Location – Deck approach spans, spring line of arches. Core in arches. Where the arch meets the girder. Curtain wall.

Task 2.4– Assessment Report

AVS will prepare an Assessment Report, which will include the findings from testing as well as presenting the concrete and corrosion into graphical exhibits and heat maps indicating the depth and severity of deficiencies discovered for all members with significant deteriorations. They will provide recommended repair strategies. (Due in November 2016)

Task 2.5 – PS&E Review

AVS will review the entire PS&E package and provide comments. (Available when needed.)

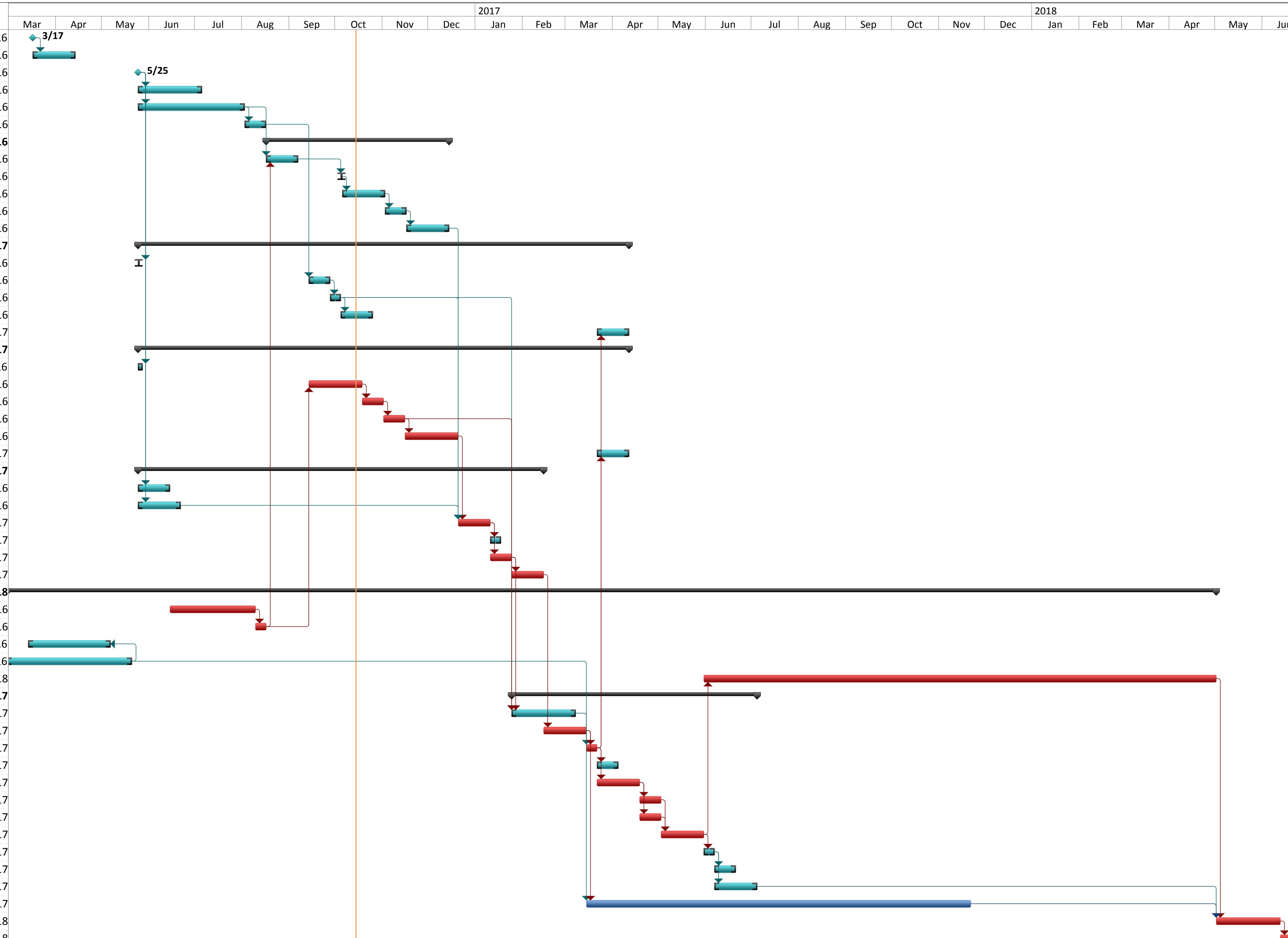
Alta Vista

October 5, 2016

Option	Description	Note	Cost
Base	Visual Inspection / Investigation 120 GPR Readings (Deck, hanger columns and arches) 4 Strength Cores (2 in spans 1 & 4) 10 borescope readings (1 in each span, 3 in hanger column, 3 in arch)	Current Scope of Contract	$47,000
A	8 additional cores - (2) in spans 2 &3, (2) in arches, (2) in hanger columns, and corresponding NDT	Additional Cores taken from accessible locations	$8,000
B	8 additional cores - (1) in each exterior girder of each span,	Additional Cores taken from less accessible locations	$30,000
C	PR Campaign	TBD	TBD
D	Additional GPR	Columns, exterior girders, abutments, etc.	$4,800

SOLANO COUNTY - STEVENSON BRIDGE ROAD BRIDGE REHABILITATION SCHEDULE

ID	Task Name	Duration	Start	Finish
1	Execution of Agreement (NTP)	0 days	Thu 3/17/16	Thu 3/17/16
2	Subconsultant Agreements	4 wks	Thu 3/17/16	Wed 4/13/16
3	Kick-off Meeting	0 days	Wed 5/25/16	Wed 5/25/16
4	Survey - Phase 1	6 wks	Wed 5/25/16	Tue 7/5/16
5	Right of Entry	10 wks	Wed 5/25/16	Tue 8/2/16
6	Survey - Phase 2	2 wks	Wed 8/3/16	Tue 8/16/16
7	**Structure Assessment**	**86 days**	**Wed 8/17/16**	**Ved 12/14/16**
8	Visual Inspection	3 wks	Wed 8/17/16	Tue 9/6/16
9	Str. Assessment Mtg.	1 day	Wed 10/5/16	Wed 10/5/16
10	Risk Based Investigation	4 wks	Thu 10/6/16	Wed 11/2/16
11	Strength Testing	2 wks	Thu 11/3/16	Ved 11/16/16
12	Assessment Report	4 wks	Thu 11/17/16	Ved 12/14/16
13	**Hydraulics**	**230 days**	**Wed 5/25/16**	**Tue 4/11/17**
14	Site Recon.	1 day	Wed 5/25/16	Wed 5/25/16
15	Hydraulic Analysis	2 wks	Wed 9/14/16	Tue 9/27/16
16	Scour Analysis	1 wk	Wed 9/28/16	Tue 10/4/16
17	Draft Hydraulic Study Report	3 wks	Wed 10/5/16	Tue 10/25/16
18	Final Hydraulic Study Report	3 wks	Wed 3/22/17	Tue 4/11/17
19	**Geotechnical**	**230 days**	**Wed 5/25/16**	**Tue 4/11/17**
20	Recon. / Permitting	3 days	Wed 5/25/16	Fri 5/27/16
21	Subsurface Exploration	5 wks	Wed 9/14/16	Tue 10/18/16
22	Lab Testing	2 wks	Ved 10/19/16	Tue 11/1/16
23	Geotechnical Analysis	2 wks	Wed 11/2/16	Tue 11/15/16
24	Draft Foundation Report	5 wks	Ved 11/16/16	Tue 12/20/16
25	Final Foundation Report	3 wks	Wed 3/22/17	Tue 4/11/17
26	**Preliminary Engineering**	**190 days**	**Wed 5/25/16**	**Tue 2/14/17**
27	Basis of Design	3 wks	Wed 5/25/16	Tue 6/14/16
28	Structure Modeling	4 wks	Wed 5/25/16	Tue 6/21/16
29	Retrofit Strategy	3 wks	Ved 12/21/16	Tue 1/10/17
30	Project Report	1 wk	Wed 1/11/17	Tue 1/17/17
31	County Review	2 wks	Wed 1/11/17	Tue 1/24/17
32	30% Roadway Design	3 wks	Wed 1/25/17	Tue 2/14/17
33	**Environmental Clearance**	**567 days**	**Mon 2/29/16**	**Tue 5/1/18**
34	Swallows Leave	40 days	Wed 6/15/16	Tue 8/9/16
35	Tree Trimming	1 wk	Wed 8/10/16	Tue 8/16/16
36	Environmantal Assistance	2 mons	Mon 3/14/16	Fri 5/6/16
37	NEP / CEQA	3 mons	Mon 2/29/16	Fri 5/20/16
38	Permitting	12 mons	Wed 5/31/17	Tue 5/1/18
39	**PS&E**	**115 days**	**Wed 1/25/17**	**Tue 7/4/17**
40	Bridge Design	6 wks	Wed 1/25/17	Tue 3/7/17
41	Roadway Design	1 mon	Wed 2/15/17	Tue 3/14/17
42	50% Plans	1 wk	Wed 3/15/17	Tue 3/21/17
43	County Review	2 wks	Wed 3/22/17	Tue 4/4/17
44	Independent Design Check	4 wks	Wed 3/22/17	Tue 4/18/17
45	Special Provisions	2 wks	Wed 4/19/17	Tue 5/2/17
46	Quantities /Estimate	2 wks	Wed 4/19/17	Tue 5/2/17
47	90 % PS&E	4 wks	Wed 5/3/17	Tue 5/30/17
48	QA / QC	1 wk	Wed 5/31/17	Tue 6/6/17
49	County Review	2 wks	Wed 6/7/17	Tue 6/20/17
50	Final PS&E	4 wks	Wed 6/7/17	Tue 7/4/17
51	Right of Way Acquisition	9 mons	Wed 3/15/17	Tue 11/21/17
52	Advertise	6 wks	Wed 5/2/18	Tue 6/12/18
53	Construction	2 wks	Wed 6/13/18	Tue 6/26/18



Project: Stevenson Schedule rev7
Date: Sat 10/15/16

Task	Summary	External Milestone	Inactive Summary	Manual Summary Rollup	Finish-only	Critical Split
Split	Project Summary	Inactive Task	Manual Task	Manual Summary	Deadline	Progress
Milestone	External Tasks	Inactive Milestone	Duration-only	Start-only	Critical	